August 10, 1933

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Engineering News-Record

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Hydraulic Model Tests for Boulder Dam Spillways
By E. W. Lane

Wood Borers Damage Building Piles
By C. A. P. Turner

Russian Railway Bridge Over Dnieper River
By C. L. Christensen

Survey of Exit Practices in Buildings
By H. B. Houghton and J. H. Courtney

Discharge Formulas for Record Western Floods
By Paul V. Hodges

Ohms of Resistance Measure Concrete Curing
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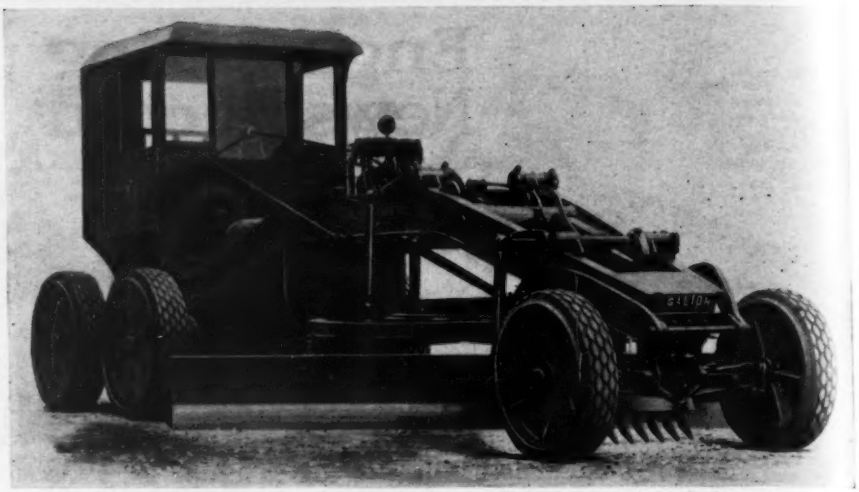

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Hydraulic-Model Tests for Boulder Dam Spillways

Design for side-channel spillways with drum gates determined after exhaustive tests—Record size models on 1:20 scale use flow of 112 sec.-ft.—Conditions of tunnel discharge studied

By E. W. Lane
Research Engineer, U. S. Bureau of Reclamation

SPILLWAY-MODEL TESTING of unusual magnitude and scope extending over a period of two years and using a special field laboratory that permitted flows of 112 sec.-ft. with a 30-ft. drop has resulted in determining the final design for the two Boulder Dam spillways. They will be of side-channel type designed to handle 200,000 sec.-ft., each discharging into the outside diversion tunnels. The extensive research program carried out by the U. S. Bureau of Reclamation for Boulder Dam was outlined by Dr. Mead in *Engineering News-Record*, Dec. 15, 1932, p. 724. This article is a more detailed account of the spillway investigation.

With the dam raising the water surface about 600 ft. above the Colorado River and with a storage capacity of 30,500,000 acre-ft., the necessity for an accurate and conservative spillway design is obvious. In addition to the 9,500,000 acre-ft. of storage that will be reserved for ordinary floods and the 125,000-sec.-ft. capacity of the needle valve and power house outlets, a spillway capacity of 400,000 sec.-ft. is being provided. The greatest flood providing any useful data is that of 1884, which had an estimated discharge of 250,000 to 300,000 sec.-ft. Probability studies indicate that the average interval between floods of corresponding peak flow would be 200 years, but on the basis of volume of flow the interval would be in excess of 1,000 years. With the operation of Boulder Dam reservoir, the peak discharge of this flood would be reduced to 75,000 sec.-ft. Because of silting, however, and to be conservative, a 400,000-sec-ft. spillway capacity was adopted.

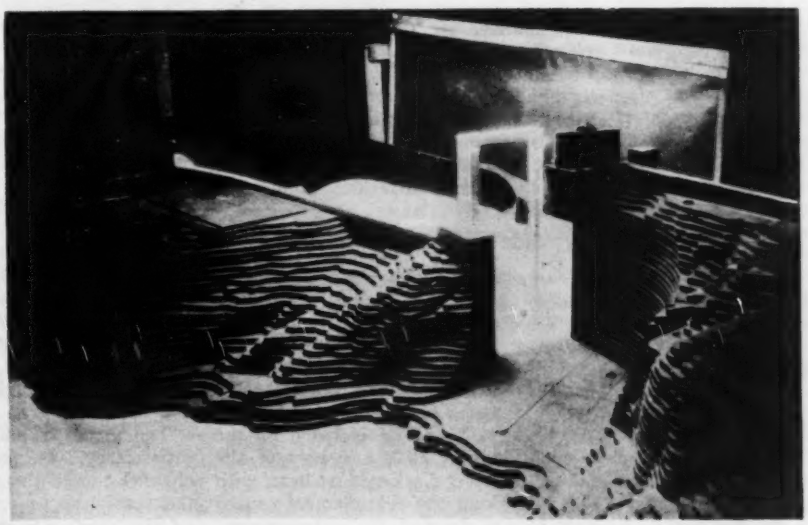

Fig. 1—Model of side-channel spillway with stoney gate at end. The model is built of sheet metal over a wood frame with boards cut to contour lines simulating the canyon walls.

With the spillways and needle-valve outlets operating at full discharge, there would be a drop between water surface in the reservoir and the river below the dam of about 500 ft. In this fall the energy of the flow over the spillways would be 22,700,000 hp., which, by comparison, is more than seven time the magni-

Fig. 2—The Nevada spillway nearing completion. From top to bottom the spillway is about 120 ft. deep.

tude of Niagara Falls (170-ft. fall with 164,000-sec.-ft. discharge). To handle safely so much potential power obviously involves careful study. Velocities also are beyond the range previously handled; in the final design velocities as high as 175 ft. per second, or practically 2 miles a minute, will be reached. Previous experience has shown that if there is no turbulence, water at high velocities can be passed over concrete with no ill effect. The design of the spillways involved the development of a form that would cause the flow at high velocities to take place with as little disturbance as possible.

For obvious reasons it was not feasible to make the spillway over the crest of the dam, and the practical layout involved discharge through the canyon walls, returning the water to the river some distance below the dam. By building one spillway on each side of the river, they could be constructed to discharge downward into the large diversion tunnels required during the construction period. This result could be accomplished at a reasonable expense. It is assumed that the reader is familiar, in general, with the arrangement. A map of the dam and appurtenant works appeared in *Engineering News-Record*, Dec. 15, 1932, p. 701.

Nearly all of the small-scale experiments of this investigation were carried out at the hydraulic laboratory of the Colorado Agricultural College, Fort Collins, Colo. (*Engineering News*, Oct. 3, 1913, p. 662). Its well-equipped shop and the shops of the college were generously made available by the college authorities. The Bureau of Reclamation was very fortunate in securing the use of a laboratory so well adapted to its needs within a short distance of its designing headquarters, and the cooperation of the college and the U. S. Bureau of Agricultural Engineering in this work has been much appreciated.

Side channel with free crest

Side-channel type spillways were adopted as best fitting Boulder Dam conditions. The theory of this type of Spillway in which the flow over the crest discharges into a channel roughly parallel to the weir was developed by Julian Hinds (*Trans.* Am.Soc.C.E., Vol. 89, 1926, p. 881) and is based on the assumption that all the energy of the falling water is dissipated as heat, with the flow along the side channel caused only by the slope of the water surface within the channel itself. In all, ten models were built and studied ranging from 1:100 to 1:20 ratios and representing five different designs. The first design of a side-channel spillway for Boulder Dam had a free ogee crest 700 ft. long. It would discharge 100,000 sec.-ft. with the water level in the reservoir at the top of the dam, but the channel itself was designed to carry all the water that would pass over the weir with the reservoir level at the top of the parapet wall, in which case the discharge would be 140,000 sec.-ft. The crest of this model was constructed of paraffin and the side channel of sheet metal. The transition from the side channel to the inclined shaft was built of cement mortar cast around an accurately constructed core.

The model functioned satisfactorily, and a great many observations were made to check the theory of the side channel. In general, the results conformed remarkably well to the theory. Extensive experiments were also made to investigate the possibility of using vanes on the crest, bottom or sides of the channel to direct part of the water passing over the crest down the channel. This would eliminate some of the impact loss and cause a higher velocity of flow in the channel, making feasible a smaller section. The studies showed, however, that the reduction of channel size obtained by use of vanes did not justify the added cost.

Side channel with stoney gate

With the decision to increase the spillway capacity to 200,000 sec.-ft., it became evident that the cost of a side-channel design with free crest would be excessive because of the great length and the topography of the canyon. To reduce the increase in cost, gate control for part or all of the discharge was studied. The prototype of the first model tested consisted of a side channel with a free crest 400 ft. long, slightly curved in alignment, with a stoney gate 60 ft. wide and 80 ft. high at the upstream end. The top of the gate was at the level of the spillway crest and at low discharges acted as a weir. For discharges greater than the combined capacity of the free crest and the top of the gate, the gate would be raised sufficiently to take care of the excess. The model of this spillway was constructed of sheet metal over a wood frame. The crest was constructed of wood, and the transition between the side channel and the inclined pyralin pipe shaft was made of wood staves accurately fitted together and held in place by outside collars. The rock of the canyon walls was simulated by boards built up in layers cut to fit the contours (Fig. 1).

The action of this spillway was not satisfactory. The curvature of the side channel caused the water to pile up on the rear wall near the upper end and on the weir side near the lower end. Many small changes were tried to im-

prove flow conditions, such as corrections in the entrance channel to the stoney gate, straightening the channel somewhat and installing baffles to dissipate the energy of the water passing over the crest. Although some of these changes resulted in slight improvements, the conditions remained unsatisfactory. It is believed that a major improvement in the action could have been secured by making the spillway channel straight, but such a design would not have fitted the topography, which would have added to the cost. Further, detailed study showed that, although the site on the Arizona side was suitable for this type of spillway, it was undesirable geologically and the stoney gate design was abandoned.

Field laboratory at Montrose, Colo.

Because of the high velocities involved in the final spillway operation, it was highly desirable to test them with as large a scale as possible to insure translation of results. Colorado Agricultural College laboratory did not readily permit the construction of a large model of the entire spillway and discharge tunnel. As no laboratory was available to provide adequate head and water supply on the scale desired, one was constructed on the South Canal of the Uncompahgre Irrigation Project near Montrose, Colo., where a steep chute provided a drop of 30 ft. and a discharge of more than 100 sec.-ft. (Fig. 3). The water supply was taken out of the ditch through two 48-in.-diameter gates into a measuring weir channel 12 ft. wide and about 75 ft. long containing baffles to quiet the flow. At the lower end of this channel was located the measuring weir, below which the water turned at right angles, passed through another set of baffles into the expanding forebay and thence over the crest of the spillway model.

First drum-gate model

The third type of side-channel spillway designed had a straight crest mounting four 100-ft. drum gates with a rise of 16 ft. The depth of flow over the crest was 26.3 ft. for a discharge of about 200,000 sec.-ft. At the new field laboratory a model of this spillway was constructed on a linear scale of 1:20. With this scale the discharge corresponding to the design capacity was 112 sec.-ft. The model was located in a bulkhead across the end of the forebay, which was fashioned to represent the topography of the canyon wall at the site. It was built of sheet metal over a wood frame and was about 20 ft. long. So far as known, this model and the one of the final design, which is somewhat longer, are the largest spillway models ever constructed for experimental work. The flow of 112 sec.-ft. and the gross fall of about 35 ft. represented about 445 hp., which is probably the largest amount of power utilized in hydraulic-model testing.

From the spillway channel the water passed through a wood-stave transition section into the inclined tunnel, which was also built of wood-stave pipe. At the bottom of the incline was a vertical bend cast in concrete followed by the horizontal tunnel represented by a 30-in. wood-stave pipe. Because the curve in the tunnel on the Nevada side of the dam site would cause disturbances in the flow that would not occur in the shorter straight tunnel of the Arizona side, the model was built to represent the Nevada spillway. Except for the bend in the Nevada tunnel and a difference in the length of horizontal section, the spillways are practically reversed duplicates.

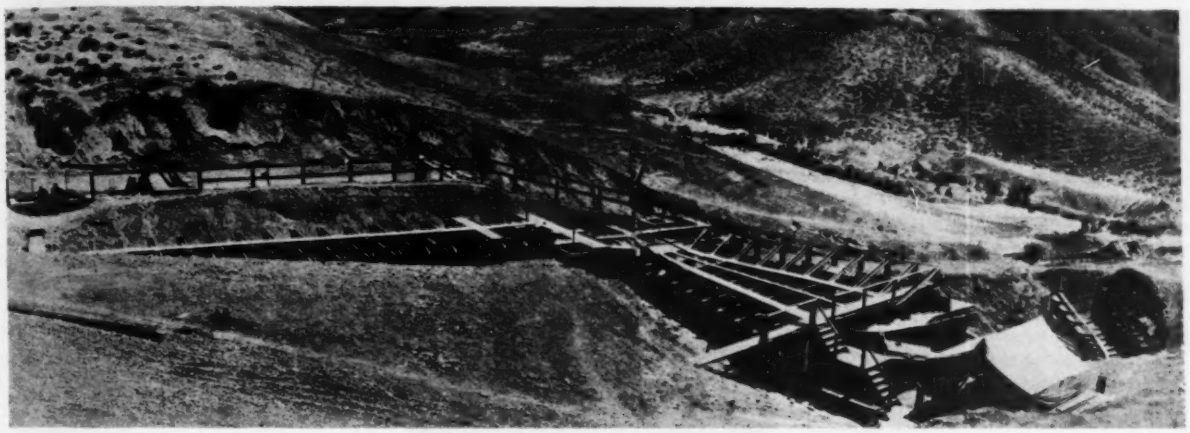

Fig. 3—Specially constructed laboratory and 1:20 scale spillway model on the Uncompahgre project, where flows up to 112 sec.-ft. were used.

To save excavation, design of the floor of the side channel was inclined rather steeply, which resulted in a high longitudinal velocity of flow with corresponding shallow depth. The water falling over the weir impinged upon a small depth of channel flow and produced a wave of considerable height against the opposite wall. At the upper end of the channel the flow was so shallow that at low discharges the channel floor was swept clear and a high wave like a surf breaker formed against the rear wall. To increase the depth of flow in the channel and thus reduce the height of this wave, the slope of the bottom was decreased. This resulted in some improvement. Further, a baffle in the form of a coping extending out from the rear wall to prevent the wave from rising too high was investigated. This proved fairly successful, more so than any other form tried in this model.

A large number of various types of channel baffles were studied. One form presented a cylindrical concave face toward the flow of the water, tending to deflect it back toward the weir side of the channel. This type of a baffle was tried (1) in a continuous row down the bottom of the channel, and (2) in various sections of shorter length in several different positions. At low flows most of the baffles produced smoother flow down the channel and inclined tunnel, but at higher discharges they obstructed the flow and for the maximum discharge would have required a larger channel cross-section.

To quiet the flow in the spillway channel and at the same time produce a less disturbed flow down the inclined tunnel, the cross-section at the top of the tunnel was contracted to produce orifice action at high flows. This resulted in greater depth and corresponding quieter flow in the channel for high discharges and smoothed out the disturbances in the flow down the inclined tunnel. Downstream from the contracted section the top of the tunnel was opened to the air to eliminate any vaccum. For this set-up the portal of the inclined tunnel was also given a bell-mouth to improve entrance conditions.

To test various forms of transition between the trapezoidal channel section and the circular inclined tunnel, a small 1:100 scale model was also constructed similar to the larger 1:20 unit. However, for this general shape of spillway no form was found to give sufficiently smooth conditions of flow on the 1:100 scale, and further studies were initiated at the Colorado Agricultural College laboratory on a 1:60 scale model.

The second drum-gate model also had four 100-ft. gates along the crest, but

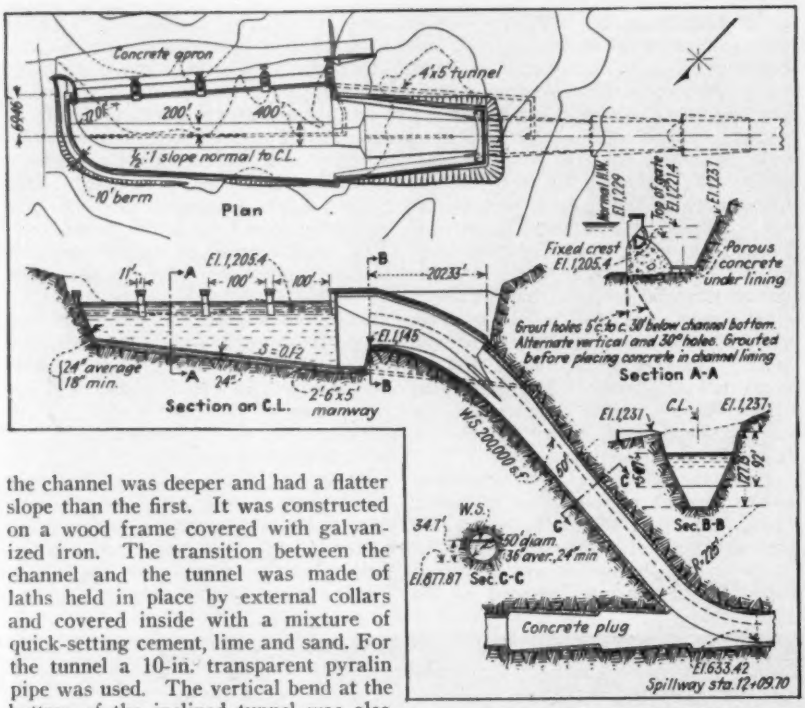

Fig. 4—Final design of the Nevada spillway as developed from model tests.

the channel was deeper and had a flatter slope than the first. It was constructed on a wood frame covered with galvanized iron. The transition between the channel and the tunnel was made of laths held in place by external collars and covered inside with a mixture of quick-setting cement, lime and sand. For the tunnel a 10-in. transparent pyralin pipe was used. The vertical bend at the bottom of the inclined tunnel was also of pyralin.

As first constructed, the tunnel portal was located just below the downstream end of the drum-gate section. The flow at the last gate caused a disturbed condition in the water entering the transition section, giving rise to an undesirably rough flow down the tunnel. When a portion of the downstream gate was blocked off, the water flowing down the channel became quieter before reaching the portal and gave a better flow condition down the inclined tunnel. To get this beneficial effect, a 60-ft. section was added to the channel between the last drum gate and the tunnel portal. This change improved flow conditions in the tunnel and was used in subsequent tests.

To increase the depth of channel flow and thus provide a greater volume of water in which the energy of the overfalling water could be dissipated, weirs of various heights were built across the channel at the portal. These devices also tended to distribute the flow more evenly at the tunnel entrance and considerably improved the flow in the spillway channel. The higher the weir the greater the improvement, but too high weirs backed the water up in the channel and reduced the discharge over the spillway crest.

The last major change in spillway design was the placing of an offset in the weir side of the channel at the end of the overflow section. This had an effect similar to the weir across the bottom of the channel and also made the side of the channel downstream from the overfalling sheet an approximate continuation of the outside surface of the sheet, which formed the boundary of the water flowing longitudinally in the channel. Without this offset in the channel wall, there was a sudden widening of the space occupied by the water flowing along the channel on the weir crest side at the end of the overfall section, which caused a lowering of the surface on that side and an unequal flow entering the transition. The final spillway design, as developed for the Nevada side of the river, is shown in Fig. 4.

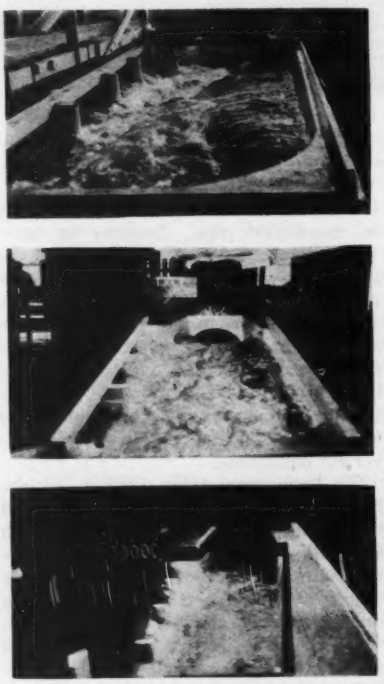

Fig. 5—Comparison of full-capacity tests (200,000-sec.-ft.) on Nevada spillway models of various scales: top, 1:20; center, 1:60; and bottom, 1:100.

A great variety of baffles and other devices were tried in the channel to reduce turbulence and to improve flow conditions in the spillway and tunnel. In all, about 100 different arrangements were tested. Some of these were quite effective in accomplishing this result, but the added expense of the concrete work to install them was not believed to be justified by the results obtained.

The form developed in the 1:60 model, just described, was duplicated on a second 1:20 scale at the Montrose laboratory. An extensive series of tests was made to compare the action of the two sizes of models, and the results obtained were very similar.

Final-design spillway models

As a result of experiments to develop the most efficient form of ogee crest, a spillway differing slightly from the second 1:20 model in the shape of the crest was adopted for the final design. Models on ratios of 1:20, 1:60 and 1:100 were made of this last design, and extensive observations were carried out to compare the results.

A close agreement was observed in the action of the models, not only in the major features of flow but also in the details of the wave formations (Fig. 5). Comparisons on a quantitative basis, both of the flow in the channel and the discharge over the weir, gave excellent agreement of results for the three sizes, warranting the confident prediction that the prototype will perform in a manner similar to the models.

Experiments on flow in the tunnel

As the velocity of water in the tunnel would reach a maximum of 175 ft. per second, it was important to know the conditions of flow in passing around the bends. The performance observed in the transparent bend joining the inclined and horizontal tunnels of the 1:60 models was encouraging. The radius of the center line of this bend corresponded to 150 ft. in the prototype. The water followed smoothly around the bottom of the bend, but some roughness resulted downstream; similar results were produced in the 1:20 scale model. A model on the 1:60 scale was built with a radius corresponding to 200 ft., which gave somewhat better results. Various larger radii were tried by means of a 10-in. rubber pipe that could be bent to any desired radius. No improvement was noted for radii greater than 200 ft., and as the flatter bends were more expensive to construct, this radius was selected.

Extensive experiments were also carried out to determine the best form of bend in the horizontal tunnel. The action of a sharp bend was demonstrated by using a 22½-deg. section of transpar-

ent pyralin bend previously mentioned. This had a center-line radius corresponding to 150 ft. and caused the water to flow around the bend in a spiral, passing from the bottom of the pipe to the top and down the other side, with great turbulence in the straight section downstream. The radius that fitted best with the arrangement of the other features of the Boulder Dam development was 819 ft. A model of this radius was built of steel pipe, with one side cut out so that the flow might be observed, and gave much better results than the sharp bend. The water rode up the outside of the bend practically to the top of the pipe but did not cross and flow down the opposite side. Experiments on bends of larger radii and compound curves gave no better results than the simple curve with 819-ft. radius, and it was therefore adopted.

At full discharge of the 1:20 model the water filled the horizontal pipe representing the tunnel to a depth of about 85 per cent at a point near the lower end. With several of the variations in the first model, a severe vibration occurred at this point. This would be extremely undesirable in the prototype, as the shocks in the prototype would be many thousand times as severe as the model. The better variations of the first model were nearly free from this effect, and it did not occur with the final design.

To obtain the greatest possible discharge over the weir crest, an extensive series of experiments was made to develop the shape having the highest discharge coefficient. The shapes of nappe for weirs with various upstream faces were determined under a wide range of velocities of approach. A form was developed suitable for a drum gate, which gave considerably higher discharge capacity than was obtained with the first designs. Observations were also made of the pressures acting on the weir under numerous conditions and of the pressures on the drum gates for various positions of the gate and depths of overflow. It is expected that a detailed report will be prepared and published at a later date covering in detail the results of the experiments on all phases of this spillway investigation.

The construction and testing of the hydraulic models were carried out under the direction of the author, assisted by R. R. Randolph, assistant engineer (Panama Canal), J. E. Warnock, associate engineer, and others. Designs of the spillways were prepared by D. C. McConaughy, senior engineer, under the direction of B. W. Steele, engineer of dams. All designs and investigations of the Bureau of Reclamation are under the direction of J. L. Savage, chief designing engineer. All engineering and construction work is under the general direction of R. F. Walter, chief engineer, with headquarters at Denver, and all activities of the bureau are under Dr. Elwood Mead, commissioner of reclamation, Washington, D. C.

Wood Borers Remove Piles from Building Foundations

Loft building in St. Paul, Minn., settles dangerously because wood borers eat wood piles out from under wall and column footings

By C. A. P. Turner
Consulting Engineer, Minneapolis, Minn.

ABOUT three years ago, when, in an investigation of settlement, some of the foundations of a six-story building in St. Paul, Minn., were uncovered, it was found that the wood piles which had originally supported the wall and column footings had disappeared. There remained for 4 or 5 ft. down only round holes whose sides had caved but little. Following this discovery the city building department issued notice that the building was unsafe for its rated loading. Upon this notice from the building department, the lessee of the building, who was not occupying it but was paying rent under lease, notified the owner that the lease was surrendered and rent payments were discontinued. A law suit followed.

As a result of this law suit, the procedure of which will not be discussed, investigations were made and testimony given which disclosed a cause of pile failure entirely new and puzzling to the writer. The facts are therefore presented here in some detail.

Building structure

The building as constructed in 1886 was four stories high and 100x125 ft., on a corner lot, and had one party wall. It had bearing walls (basement limestone and brick above) and interior cast-iron columns spaced 14x19 ft. and carrying 14-ft. steel girders, wood floor joists and floors, with plaster ceilings. In 1912 two stories were added. The structure was planned for light manufacturing. The original column footings were carried on clusters of nine piles 14 to 16 in. in diameter and probably 40 to 50 ft. long. Further details will be given later.

The ground occupied by the building was a part of a swamp bottom of clay. Fills across this bottom had previously been made for the two streets bordering the building lot. These fills were unsettled. A part of the lot area was to some extent a dump, while the clay bottom came to the surface in other parts. In constructing the foundations the fill material stood up well, and where the sand and gravel of the fill had spread into the clay there was a layer of hard digging. Elsewhere the clay bottom was soft. In these materials the column footings were built as follows:

The piles were cut off 4 or 5 in. above the existing ground surface. Two-man flat stones were set on the pile tops, and the spaces around them were filled with spawl and the capstones wedged with spawls until they bore nicely on the pile tops. The top of this dry masonry was smoothed up with mortar, and on this a masonry pier about 5 ft. square was laid up in natural cement mortar to the basement floor 8½ ft. above. A 3-ft. square capstone was set on each pier to carry the cast base for the column. The piers were filled between with sand, gravel, stone chippings, etc. Sleepers were laid on this fill and floored with spiked plank. The sleepers appeared to have been embedded in natural cement and about 1¼-in. layer of cement placed under the planking. The total load on the footings is estimated to have been 1¼ tons a sq.ft.

When in 1912 two stories were added to the building it was observed that the settlement had been 2 or 3 in. at one end and about the same amount at some of the interior columns. This was attributed to drawing down the water by sewerage and other city improvements and consequent shrinkage of the clay. With the added two stories there was no immediate change in elevation, but gradual settlement soon began and reached from 3 to 5 in. additional at some of the columns, which led to the investigation being described.

The uncovered foundations disclosed, as previously stated, that many piles had disappeared for several feet down, leaving uncaved holes in the clay. The clay for a few inches annularly around the hole was dry but farther in was soft and plastic. Also the clay under many of the footings was so plastic that it could be squeezed, like putty, into a ball. It was so moist in fact that it seemed impossible that the piles would rot were they well sealed at the top.

This fact and the further fact that so little remained of the piles indicated that they had been eaten out by borers. Had rot alone been the cause, the decayed pile would be more or less intact, filling the hole. As during the last five years there has been much less rainfall than for many years previously, the soil under the building would be dry and allow beetles to get under the floor and at the piles. This was shown to have been the case.

By getting hold of a piece of a pile with a chain and breaking it loose down below, samples were pulled up from a depth of 4 to 6 ft., which showed fresh pine timber in the middle of the outer surface scored by these borers. Ap-

parently their mode of operation, as might be judged from the brown wood rotted remains of some other piles near the top, was to attack the pile 1 or 2 ft. down from the head, eat it away completely for some length and working from the outside toward the center, leaving the dirt of the worms with the little chips they would bite out of the pile in strings. This is quite different from their work in dead and down timber in which they seem to bore and tunnel the wood more like the teredo. Here they worked around the periphery from the outside toward the center, the grub scores following closely but not exactly the direction of the grain.

As regards the mode of attack of the foundations, the beetles undoubtedly laid their eggs on the exterior surface, which was just damp enough to suit the borers. Starting in perhaps 2 ft. down below the top of the pile, which was a little too dry to make nice chewing, they cut off the moisture that previously had been brought up to the top of the pile by capillary attraction; as a result the extreme top rotted into brown punk, which in the empty holes had dropped down and broken up about the upwardly projecting core lower down.

It seemed somewhat dangerous to dig deeper and extract a section on which the grubs were still working, but finding the worm holes indicated the reason why the top portion had rotted without showing grub marks. The wood borers, which are such a pest in apple culture, hatch from the larvae from the *saperda condida* and the *chrysobothrid femorata* beetles, which had deposited their larvae 2 or 3 ft. down from the top of the pile, as the pile at that point was a little more moist than at the top and hence more succulent to chew up.

Evidence on dry-wood borers

Because the disappearance of the top 4 or 5 ft. of the piles was attributed by most of the engineering witnesses in the law suit to rot rather than to borers, the writer reviewed literature in various engineering treatises, including Merriman, Lanza, Rankine, *Civil Engineering*, Trautwine, etc., and was surprised to note how little information was presented on the subject. Thus in our standard engineering pocketbooks, including the old edition of Kent, the most important cause of timber decay is not mentioned.

Dry and fairly well-seasoned timber is attacked by small borers that cause powder post. Dead and down timber in the forest is attacked by various wood borers. Living timber suffers to a lesser degree. There are many species of borers that eat into the bark of certain special varieties of timber. Others only attack the hard wood or inner portion. These wood borers in the early stage of their development belong to that class of insects known as *coleoptera* or beetles in their final development. More than 80,000 varieties have been described by entomologists, although fortunately the varieties that attack timber are more limited. All of these insects develop from the egg into worms, which, when they attain full growth, incase themselves in a shell and, following a period of repose, change into the developed beetle. Certain varieties of these beetles prefer specific types of growing timber, and when this kind of timber covers large areas an epidemic of the attacking beetles may occur. A million and a half trees in the Harz forest in Germany were destroyed in a single season by two varieties of these beetles.

The most active of the wood borers are those that constitute the pest of the apple grower known as *saperda condida* and *chrysobothrid femorata*, which appear to attack the wood where the bark is broken and in any portions where dead timber is exposed. The eggs of the beetles are eaten and destroyed by black ants, birds, etc., so that they do not multiply as rapidly as under conditions that are more favorable to them.

The development of the borer is more rapid where the temperature is from 60 to 70 deg., and at freezing temperature they are practically dormant. Apparently the time in which the borer will work before changing into the beetle may vary from two to three years, as the conditions are those of outside exposure in winter and summer seasons down to a much shorter time, where the temperature is more constant, so that where the latter is the case practical destruction of the timber may be accomplished in a period as short as one to one-half years.

The safety of the piling in conditions as above described depends on complete sealing of the top of the pile from access by these destructive insects. Such sealing is accomplished by present-day concrete footings more satisfactorily than with the old-fashioned rubble-stone capping and piers that were built on top of the piling column in the '80's.

The rectification and strengthening of the foundation where piling has been thus eliminated resolves itself into a very simple matter where the clay will harden as it dries out. The damaged piling may then be burned out, and the heat of its removal in this manner will stiffen the clay so that concrete may be safely deposited with reinforcement and the structure placed on an even more stable basis than originally at a relatively small expense. This method of repair would be permissible only where the material surrounding the pile holds sufficient clay to harden satisfactorily under heat. Thus the equivalent of four to five months' rent would have been ample under competent engineering supervision to have made the footings safe and rendered the structure in its admirable location one of the most desirable loft buildings in the city, and incidentally would have obviated the dispute between lessor and lessee involving many times that cost.

Russia Builds Rail-Bridge Over Dnieper River

By C. L. Christensen
Formerly Designing Engineer, Niagara River Arch, Michigan Central Railroad

RUSSIAN METHODS of bridge design and construction are well illustrated by a railroad structure over the Dnieper River at Dniepropetrovsk (about 50 miles upstream from the Dnieprostroy power station), which was completed recently. The bridge is about 5,350 ft. long, of which 4,600 ft. is of reinforced-concrete construction.

Fig. 1—Under a load of two locomotives the 170-ft. spans showed an average deflection of $\frac{1}{4}$ in.

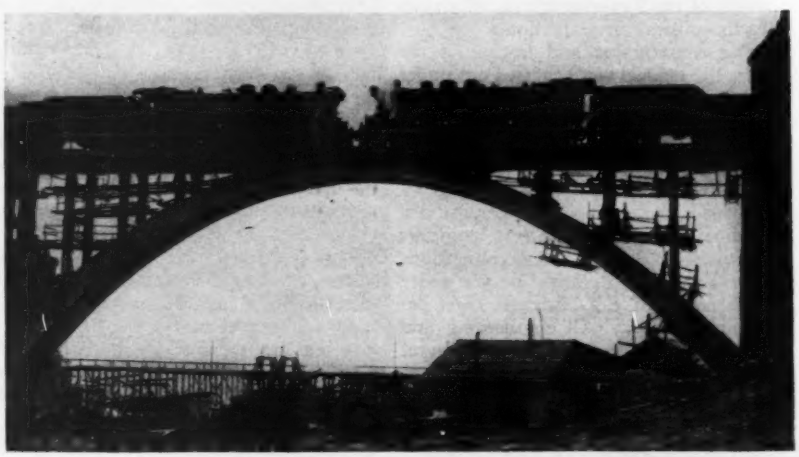

Fig. 2—In pouring the 170-ft. spans both steel and timber truss centering was used. By means of a land concrete plant and a cableway, supplemented by a floating concrete plant, all arches were poured simultaneously.

consisting of fourteen 170-ft. arches, fourteen 95-ft. arches, three 50-ft. arches and four 53-ft. continuous-girder spans. The remainder of the length is made up of two 360-ft. steel through-trusses over the navigation channel. The bridge accommodates a single-track railway and two sidewalks.

At the bridge site the river is divided by an island into a main and side channel. The bridge crosses the main channel in a straight line, but on the island and across the side channel it is on a curve, to facilitate the track layout along the river bank. During the first years of the World War the piers were built in both channels for a bridge consisting mainly of 360-ft. steel truss spans, but the superstructure was never erected. In 1931 a new design was made in which all except the two spans at mid-channel were reduced in length by the construction of a number of new piers. Most of the piers in the river were built by the pneumatic method and were carried down to solid rock. Some few piers were built by the cofferdam method, while concrete piles were driven to support the piers on the island.

All arches consist of two ribs connected by diaphragms. In the 170-ft. arches the ribs are vertical, and the lateral forces from the trains and the wind are transferred to the tops of the piers through the floor slab. In the 95-ft. arches, however, each span is an entireiy independent superstructure on low piers, and the lateral forces are resisted by giving the arch ribs a transverse batter (Fig. 3).

Fig. 3—The 95-ft. arches of the curved approach were founded on low piers, and the ribs were battered outward to provide resistance against lateral forces of wind and train sideway.

The new piers were built during the winter of 1931-1932, and the superstructure was completed during 1932. The total construction period was only sixteen months. The stones for the concrete were quarried on the rocky river shore, transported by cableway to the island and crushed there. The falsework trusses for the river spans were built as three-hinged arches, some of timber and some of old steel trusses, rebuilt and reinforced at the construction camp. In order to finish the concrete work before winter, and thus avoid expensive and difficult cold-weather concrete work, a sufficient number of falsework trusses was made to permit all the arch spans to be built simultaneously. These falsework trusses will be used again during 1933 for a similar bridge over the Volga River. The concrete was transported from stationary mixing plants by cableway, except that the concrete for the midstream arches was poured from mixing plants on barges. The reinforcing bars in the arch ribs were spliced by welding. There were about 10,000 lap-welded splices. One construction difficulty was occasioned by the fact that the old piers built in 1914 were found to have a very poor concrete behind their fine-looking stone facing. A grouting operation of major proportions was therefore necessary to make them adequate for the loads.

When the bridge was finished, it was tested for deflections and vibrations under the load of two heavy freight locomotives. On the 170-ft. spans the average deflection was about $\frac{3}{8}$ in., and horizontal movement at the floor-slab expansion joints over the piers was $\frac{1}{4}$ in. Finally the two locomotives coupled together ran over the bridge and turned the curve a number of times at 50 miles per hour. The bridge was then opened for traffic.

Engineers and contractors

The bridge was designed by the engineering staff of Gosproektrans. The two steel spans were manufactured and erected by Stalmost. The concrete arches and piers were built by the Dniepropetrovsk department of Mostatrust. All equipment, such as floating cranes, concrete plants and cableways, was designed and built at the construction plant by the engineering staff of Mostatrust. M. A. Kienia was chief engineer of construction, while the problems of providing labor and material were handled by the general manager, N. P. Bogdanof. The writer was consulting engineer for Mostatrust during the construction.

Small Shield-Driven Tunnel Lined With Trapezoidal Blocks

Interlocking precast units of new shape used in 4-ft. drainage tunnel under Pennsylvania Railroad tracks at Norristown, Pa.—Blocks made on job in hydraulic press

A SHIELD-DRIVEN 4-ft. drainage tunnel 170 ft. long and lined with a new type of interlocking trapezoidal-shaped precast concrete block has just been completed under seven tracks of the Pennsylvania Railroad at Norristown, Pa. Cover between bottom of ties and top of tunnel averaged 14 ft. The tunnel was driven through filled ground, with driving speeds varying from 25 ft. per eight-hour shift in clay fill to 12 ft. per shift in rock fill. The driving and lining process was somewhat similar to that described in *Engineering News-Record*, Nov. 13, 1930, p. 771, the same shield (slightly modified) being used, but the shape and size of blocks and method of manufacture were materially changed. An inside view of the tunnel is shown in Fig. 1.

Formerly the lining blocks used in this system of tunneling were hexagonal in shape, 8 in. wide in the direction parallel to the tunnel line, poured in cast-iron molds and erected in staggered rings with every other block projecting 4 in. beyond the adjacent units. The new blocks are trapezoidal in shape, 8 in. wide, and $4\frac{7}{8}$ in. thick, made up in two types to interlock together in a complete circular ring forming 8 in. of completed tunnel lining 4 ft. in diameter. Eighteen blocks make up the ring, nine of the A type and nine of the C type. The two types differ only in the tongue and groove portions of the contact edges which, with the trapezoidal shape, constitute the interlocking feature of the unit. Tongues on the blocks are $1\frac{1}{2}$ in. wide and $\frac{1}{2}$ in. high; grooves are $\frac{1}{2}$ in. deep, and $1\frac{5}{8}$ in. wide, both tapered to provide a snug fit. The A block has a groove on the long end, and both edges and a tongue on the short end. The C block has a tongue on the long end, and both edges and a groove in the short end. In addition, the C block carries transverse tongues and grooves on the ends to assure proper spacing with respect to adjacent rings. Before placing, the contact edges of the block are dipped in hot-blown asphalt, which provides a watertight joint seal. Details of the blocks are shown in Fig. 3. Each block contains 0.206 cu.ft. of concrete and weighs about 32 lb. Twenty-seven blocks are required for 1 ft. of tunnel.

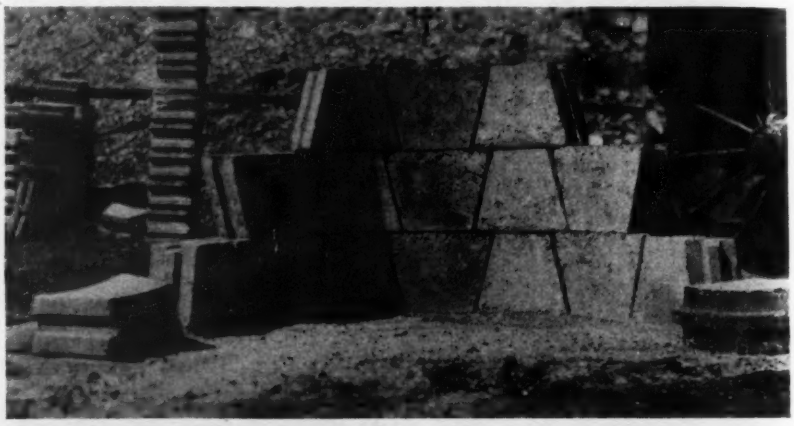

Fig. 2—Part of three rings set up to show how blocks interlock in perfect alignment. Joint seal formed by dipping blocks in hot asphalt is not shown.

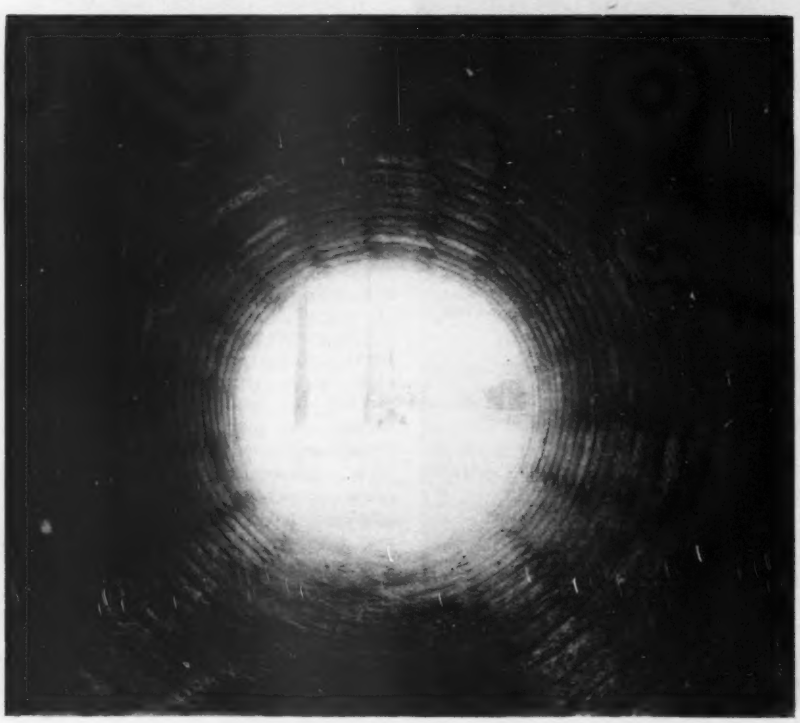

Fig. 1—Inside view of 4-ft. shield-driven tunnel lined with trapezoidal concrete blocks. Because of light conditions within tunnel, described in this article, a satisfactory interior view could not be obtained. This view is that of a similar tunnel just completed in Philadelphia, and is shown to illustrate arrangement of blocks.

Shield design

The shield is 4 ft. 10 in. in outside diameter and has an extreme length of 10 ft. $5\frac{1}{2}$ in. The skin is steel plate, $\frac{3}{4}$ in. thick, except for the tail, which is 14 in. long and only $\frac{1}{8}$ in. thick. The forward half of the shield is articulated, the diaphragm frames of the front and rear parts of the shield being joined by four directional control screws. These control screws are used for steering of the shield. The shield hood extends 21 in. beyond the lower cutting edge. Eighteen hydraulic jacks are arranged to operate in two banks of nine each, alternating jacks making up one bank. Each jack, 4 in. in diameter, is equipped with a 2-in. auxiliary plunger that carries the jack shoe. The jack shoes have a partial ball-and-socket connec-

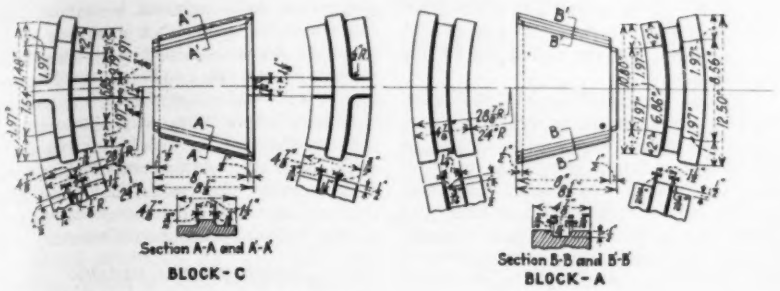

Fig. 3—Details of new trapezoidal-shaped concrete blocks that were used in small drainage tunnel.

tion with the plunger, giving the shoe an even bearing on the block. When the main jack is extended, it bears against the jack shoe collar of the auxiliary plunger, and when in this position a dog on the main jack engages the jack shoe. When retracting, the dog pulls the shoe and auxiliary plunger back to the main jack. Oil is used as a pressure medium in both large and small jacks, the shoving pressure being actuated by air supplied by a portable compressor outside the tunnel. A reducing valve and two three-way valves supply high and low pressure oil to the larger jacks. Although the equipment is designed for jacking pressures of 4,000 lb., a pressure of 1,200 lb. is usually sufficient, with 240 lb. used as the low pressure. The retracting oil pressure is supplied by an air pump floating on the air line, this pressure actng continuously.

Tunnel driving

A sheeted shaft about 14 ft. deep was sunk in the outlet portal of the tunnel. At the bottom of this shaft a manhole will be built to connect the tunnel with an exising drainage system. The shield was lowered, assembled, to the bottom of the shaft, and driving was started from the lower end of the tunnel.

At the start of the driving, the shield rested on a structural-steel cradle accurately set to line and grade. The jacks pushed the first ring of concrete blocks home against a precast reinforced-concrete starting ring, which in turn was supported by the head frame of the structural cradle.

Muck was pulled down from the heading by hand and was thrown upon an 8-in. belt conveyor, 15 ft. long, that extends through the shield. The conveyor is operated by an air motor. From the end of the short conveyor the muck was dumped into ¼-yd. steel buckets set on a flat car operating on a 12-in.-gage track. The flat car was divided into two parts, the rear part carrying two of the muck buckets and the front part consisting of a low platform upon which the lining blocks were carried from the portal to the shield. The car was handled in and out of the tunnel by hand. At the top of the shaft was rigged up a monorail hoist operated by air. The hoist could lift two muck buckets out at a time, the monorail permitting the buckets to be moved over the side of the shaft for dumping (Fig. 5).

Placing the block

The operation of placing the lining blocks is as follows: Consider the heading excavated ready for a shove of the shield. The nine alternate C block jacks are retracted, and nine C blocks, with their small end forward, are set in place against the concrete starting ring. As each block is set in place the dog on the corresponding jack is released, bringing the auxiliary plunger of that jack out against the block under low pressure, holding the block securely in place against the last ring of blocks. After the nine C blocks have been placed, the full pressure of the jacks bearing against them is released, shoving the shield ahead. The A blocks are then set in place, with the large end forward. As these blocks are set, the dogs on corresponding jacks are released, bringing the low-pressure plungers out against the blocks to hold them in place. Because of the wide end of the A block that is being placed forward, it is necessary to provide some additional clearance by shoving the shield ahead before these blocks are placed. In hard shoving the shield is pushed ahead just far enough to provide a clearance but in easy going the entire 8 in. shove of the shield is made against the set of C blocks. After the A blocks have been set, completing the ring, full pressure of all jacks is released to shove the blocks into place and maintain an even circumferential joint. After all the blocks are in place, the mucking out of the heading is resumed, and then the jacking cycle is repeated. The tunnel crew consisted of a foreman and five to eight laborers, depending upon the type of ground encountered.

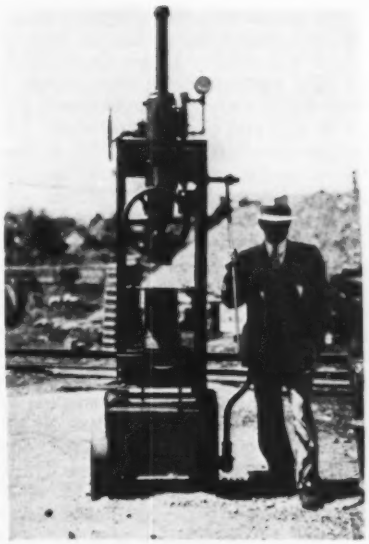

Fig. 4—Hand-operated hydraulic block press, which has a capacity of 400 blocks per day.

Fig. 5—Muck was handled at portal shaft by air-operated hoist on a monorail. Steel muck buckets were handled to and from heading on 12-in.-gage car.

The blocks were manufactured on the job in a hydraulic press. A mix of 1:1½:2 was used to obtain the 4,000 lb. per sq.in. compressive stress specified at the end of 28 days. The aggregates consist of sand and ½-in. crushed stone. The hydraulic press is shown in Fig. 4. Two sets of molds were required, one for each type of block. The concrete was mixed in a small mixer, and the amount of concrete dumped into the mold was carefully determined by weight. The hydraulic jack in the press was operated by hand. Indicators on the press showed when the blocks had been compressed to the exact required length. After compression, the top and side molds were lifted off, and the block was removed on the bottom mold and left over night before the bottom was stripped. The block-making plant had a capacity of 400 blocks in eight hours. The usual procedure was to make 200 blocks of one type in the morning and 200 of the other type in the afternoon. A crew of five was required for the operation of the block plant.

Blocks were cured for at least seven days before use in the tunnel. After curing, the contact edges were dipped in hot-blown asphalt to provid the seal. A rack was set in the asphalt container, and the top of the rack was kept just below the level of the liquid asphalt. Shoving the blocks across the rack from one end of the asphalt tank to the other was sufficient to give the block the right amount of asphalt coating.

Contractors and personnel

The tunnel was built for the Pennsylvania Railroad. The contractor was Vincent Neamand, Philadelphia, associated with the Tunnel & Mine Machinery Co., of Philadelphia, holders of patents on the shield and block.

Survey of Exit Practice in Buildings

Number, location, size and types of exits, population and areas served, travel distance to exits and obstructions found to their free use as disclosed by wide survey

By H. B. Houghton and J. H. Courtney
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IN THE preparation of minimum requirements for means of egress from buildings, the building-code committee of the Department of Commerce and the building-exits code committee of the American Standards Association considered it advisable to accumulate as many data as possible on general current practice in the design of exits. The collection of these data was considered necessary not only to supplement the information already possessed by the committee members as a result of their professional experience, but also, by including a sufficient number of cities, to secure a fairly good cross-section of current practice throughout the country. The committees decided that a survey of modern buildings carried on in as many cites as the time allotted to the work would permit would secure the desired information most efficiently. The survey was carried out by the staff of the building-code committee.

Method of survey

The survey was conducted in the office and field and covered places of assembly, public, institutional, residential and business buildings erected within the last ten years. Buildings of mixed and special occupancies were not included for the reason that these are usually considered as special cases for the building official to decide and consequently would not be subject to general requirements for exits.

The office survey consisted of the collection and tabulation of information and recommendations obtained by correspondence with architects and engineers, officials of hotel chains, code writers, school superintendents, theater executives and others having special knowledge of exit requirements. Floor plans obtained from architects and from other sources were studied. A digest of exit regulations in state labor laws, tenement-house laws and building codes was made, and the fundamental requirements of each were listed and compared. A study of fire records was made to determine to what extent exits were responsible for loss of life in fires.

In the field survey, contact was first made with the building official of each city visited. A list of representative buildings was prepared with his assistance, and plans for these buildings were secured from the files. The required information was then taken from the plans and checked by an inspection of the building. Floor plans were obtained from the rental managers of the buildings to supplement the field notes. During the building inspection notes were made concerning obstructions to exits and the marking of the exits, and figures on the population served by the exits were secured when available. Interviews were obtained with building officials, state labor officials, architects and school superintendents, and much pertinent information was obtained on exit design and the minimum requirements for exits. After completion of the field survey the material from all sources was assembled for presentation to the building-code committee.

Design practice variable

The survey did not reveal any particular uniformity in methods of exit design either in current architectural practice or in code or labor-law requirements. On the contrary, a wide variation in the fundamentals of exit design was observed in the different cities. Architectural practice differed in regard to code requirements, some buildings barely meeting the code requirements while others provided exits in excess of requirements. Some architects were of the opinion that the code regulations were safe, while others were of the opinion that code regulations were not severe enough. The need for regulation of exit design was generally recognized, however, by the architects interviewed. The building officials considered the exit provisions as among the most important of the building regulations, although some difference of opinion exists as to minimum requirements. The opinion was frequently expressed, however, that the securing of adequate exits in existing buildings is a more difficult problem than in the case of new buildings. This is particularly true in existing buildings in which a change of occupancy is contemplated.

Certain cities do not require a permit to make repairs. Under the pretense of repairs such a complete interior alteration may be effected that a one-family dwelling may be converted into a multi-family house with no change in existing exits. The existing exits may be entirely inadequate for the increased population and may result in fatalities in the event of fire. In one of the cities visited the loss of life in a recent fire was attributed directly to this cause by the building official.

Stairways and fire escapes

Current practice in the number of stairways provided in buildings of different occupancies showed considerable variation. Of the 36 office buildings inspected, 20 had only one stairway, ten of these one-stairway buildings being over ten stories high. Of the buildings of other occupancy, however, 80 per cent had two or more stairways. Stairways varied in width from 3 ft. to 10 ft., the widest being found in a department store not equipped with elevators. Schools were generally provided with wide stairways. The stairway width found most frequently was around 42 in., although a great many 36-in. stairways were observed in office buildings

and apartment houses. The stairways were generally located so as to utilize dead space or space least valuable as rentable area. This, of course, influences the distance to the exit and probably accounts for the wide range of these distances of 30 up to 200 ft. In most cases the distance was less than that allowed by the local code. Judging from the changes in location and the increase in number of exits made on plans submitted for approval, the building official is called upon frequently to exercise judgment in the location of exits.

Current practice seems to favor inclosure of stairways, although many buildings having open stairways were observed. Open stairways in schools were found quite generally and seem to meet with the approval of the school superintendents interviewed. Many department stores and hospitals had open stairways, but those of more recent construction were inclosed. The use of fire doors in inclosed stairways was quite general, although some wood doors were found. One hotel-chain executive expressed a preference for wood doors, his reason being that smoke causes more loss of life in hotels than fire and the wood door, fitting the opening more tightly than the metal door, prevents the entrance of smoke into the stairway to a greater degree.

Fire escapes were seldom observed on new buildings, although their use was quite common on the older buildings. While not favored by all the building officials, they seem to be the only solution to the problem of providing adequate exits with a minimum of expense in the old buildings. Several high buildings recently built were found equipped with inclosed spiral slide escapes. In some cases these were provided in addition to the required stairways, and in others they replaced one of the required stairways. While no evidence was obtainable in regard to the efficiency of these particular escapes, it would seem that a better lighting of the interior would make them appear a less hazardous form of descent to the inexperienced. The open type of exterior slide escape was observed on several schools and has proved efficient in fire drills, according to one building official.

Fire towers and horizontal exits were not common, the use of the latter being confined principally to hospitals. Where additions had been made recently, the new fire-resistive wing was usually connected to the older portion by a horizontal exit. Stairways that discharged directly into a street were in the minority, the usual practice being to locate them within the interior of the building and discharge to the street through a corridor or lobby. Although many codes require such corridors and lobbies to be protected, no such construction was noted on the survey. The usual practice was to utilize the corridors and lobbies for show-window display, cigar booths and shop entrances. Sprinkler protection in these locations was unusual.

SUMMARY OF SOME AVERAGE FIGURES DEVELOPED BY THE SURVEY OF BUILDING-EXIT PRACTICE

Occupancy	Type of Construction	Percentage of Buildings Having		Aver. Floor Area Served by Stairway, Sq. Ft.	Aver. Width of Stairway, In.	Aver. Max. Distance to Exit, Ft.
		1 Stairway	2 or More Stairways			
Office Buildings	Fire-resistive	55	45	8,000	45	85
Hospitals	Fire-resistive	10	90	2,700	43	77
Hotels	Fire-resistive	7	93	6,500	42	88
Dormitories	Fire-resistive		100	2,540	48	53
Apartments	Fire-resistive	15	85	4,200	42	63
Apartments	Non-fire-resistive	32	68	2,550	40	38
Schools	Fire-resistive	6	94	8,700	55	63
Stores	Fire-resistive	30	70	8,900	59	106
Factories	Fire-resistive	44	56	10,500	50	86

Obstructions and marking

As a general rule, the exits had a distinguishing mark of some character to indicate their function. Some had a legend painted on the door, others had colored lights over the door or were equipped with a lighted exit sign. Directional signs indicating the location of the exit were found very infrequently, their absence usually requiring an extended search for the exit when the building was of any considerable area. Floors were seldom designated on the interior of the stairways. This makes it difficult for persons not familiar with a building to determine when the street floor has been reached, and in the confusion incident to a fire in such buildings serious consequences might be caused by the failure to mark properly the street level. This is especially true in buildings in which the stairway extends to a basement.

Most codes contain provisions requiring maintenance of exits and prohibiting obstructions of any nature in the exits. The successful enforcement of such provisions is exceedingly difficult, according to the building officials. Ample evidence of this fact was apparent on the survey. The obstructions found included exposed radiators and piping on stair landings, projecting chimneys, incinerators and structural members in the stairway inclosures, and materials and receptacles of all sorts stored in corridors, on stairway landings and in stair halls. In many cases this storage of material had greatly reduced the width of the passageway and offered a serious obstruction to traffic in the event of fire, in addition to the fire hazard of the material itself. Self-closing fire doors were found held in the open position by the use of wedges, and in one department store the fire doors had been removed entirely at the sales basement level, possibly to facilitate traffic but certainly defeating the purpose for which they were installed. Doors leading to roofs were frequently found locked, and in one fine department store the exit leading to the exterior from the stairway serving the upper stories was securely padlocked.

L. & N. Railway Builds New Bridge Over Tennessee River

THE NEW BRIDGE over the Tennessee River at Danville, Tenn., completed by the Louisville & Nashville Railroad, consists of five 200-t. through-truss spans, a 177-ft. through-truss span, a 74-ft. through plate-girder, and a vertical-lift span of 294 ft. Warren-type trusses with verticals are used. The structure replaces an old bridge at an adjacent location and includes a number of special features, according to an article in the June 17 issue of *Railway Age*. A departure from usual practice is the introduction of collision struts between the end posts and the intersection of the hip verticals with the bottom chord. In the lift span an ingenious scheme was introduced for taking up slack or otherwise adjusting the length of the operating cables. Instead of being attached directly to structural members at the tops of the towers, the upper ends of the operating cables are secured to drums that may be wound or unwound manually in small amounts by applying a large wrench at the end of a worm that engages a gear attached to the drum shaft. To increase resistance to corrosion, the top-flange angles or coverplates of stringers and floor beams, the stringer lateral system, portals and the bottom struts of the sway-bracing are made of copper-bearing steel with a minimum of 0.2 per cent of copper. Finally, the tops of all floor beams and stringers were covered with creosoted protection timbers sheathed with galvanized metal.

The project was executed under the general direction of W. H. Courtenay, chief engineer, G. R. Smiley, assistant chief engineer, and J. M. Salmon, bridge engineer. The substructure contract was held by the Dravo Contracting Co., Pittsburgh, and the superstructure contract by the Virginia Bridge & Iron Co., Roanoke, Va.

NEW PLANS FOR THE MISSISSIPPI

Recent Investigations of Bank Revetment

Last of a Series of Eight Articles

Report of underwater survey of existing revetment—Submerged groins inadequate—Hot-mix asphalt mats laid successfully—Riprap revetment tests indicate practicability of certain types

ALTHOUGH employed for half a century to armor the scouring banks of the lower Mississippi River, and despite the fact that material advance in durability and economy has been made during the past ten years, revetment still remains a device of great cost and uncertain life. The cost and life considerations have been of increasing concern to river engineers in the last three years in which the present Mississippi flood-protection project has called for a bank-protection program beyond all precedent. Experience, it was realized, called for new planning if justification was to be provided for the enormous expenditures.

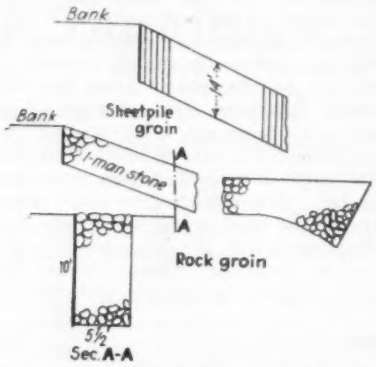


Fig. 1—Sketches indicating types of submerged groins constructed for experimental purposes on shallow stable bank and deep caving bank.

In the river south of Cairo, Ill, revetment until the last three years has been almost the only bank protection employed. Beginning with the fish-pole mats laid down in New Orleans harbor in 1878, revetment structure has progressed through various forms of brush and timber mat to the four standard types recently in use. All the older constructions have virtually ceased to exist in Mississippi practice except for occasional and special purposes as, for example, the board mats for holding banks along the lines of permeable dikes.

The four standard types of revetment now in service, some of which are still existing practice on the lower Mississippi, have been fully described in *Engineering News-Record* as follows: Framed willow mat, May 1, 1930, p. 720; fascine mat, July 2, 1930, p. 4; articulated concrete-block mat, Dec. 25, 1930, p. 996; and articulated concrete-slab mat, April 14, 1932, p. 554.

Practice in both the design and construction of all these types of revetment has been standardized and highly perfected in equipment and methods. Construction economies, it may be said with reason, have reached about the maximum possible for the types and qualities of revetment constructed. While all the types named are standard, practice in the last two or three years has shown a trend toward further simplification. Willow mats of both types are being limited in use because of increasing scarcity of materials and because they are more expensive and less durable than concrete. Of the two types of concrete mat, the articulated-block type has increased in favor until the slab type is going out of use. In the last two years articulated-block mat has been greatly improved in strength and tightness by the use of larger and heavier blocks, fewer and closer joints, heavier metal ties and corrosion-resisting wire and cable.

The expenditures for bank revetment on the Mississippi from 1881 to 1931 inclusive, according to official records, has been $89,246,930. Of this amount, $33,735,000 was spent prior to 1918, which may be taken as the year when modern revetment practice became stabilized. In this period, 1918 to 1931 inclusive, 501,790 ft., or about 95 miles, of revetment was constructed at a cost of $32,545,924; the cost per bank-foot, averaged for each year, ranged from $46 to $96, and averaged for the fourteen years $64.86. Maintenance in this period cost $22,966,006. The grand-total expenditure was therefore $55,511,930 or $584,336 a mile of bank protected. Construction expenditures during fourteen years have ranged from $242,880 to $506,880 a mile of bank and have averaged $338,200.

Bank-mile costs are indefinite because of varying widths of revetment. Compared on the basis of the square (100 sq.ft.) as a unit, the costs and assumed life periods of various types of revetment are given as in the accompanying table in a report dated May 7, 1932, to the chief of engineers by Brig.-Gen. T. H. Jackson, president of the Mississippi River Commission. The tabular costs are based generally on 1931 work, for which costs ran materially below the averages of preceding years since 1918. Monolithic-concrete upper-bank pavement will cost about the same per square as articulated-block mat of the lowest costs given in the table.

Fig. 2—Diagram of experimental plant for molding and sinking reinforced-asphalt mat revetment in New Orleans harbor.

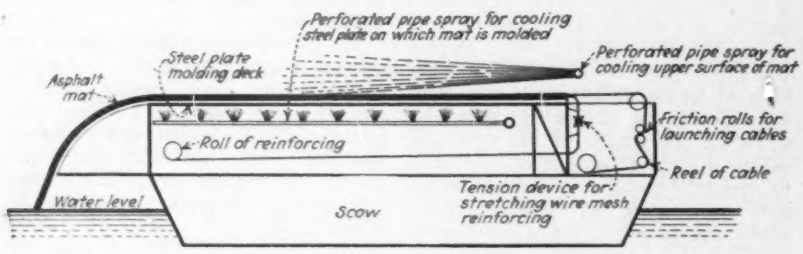
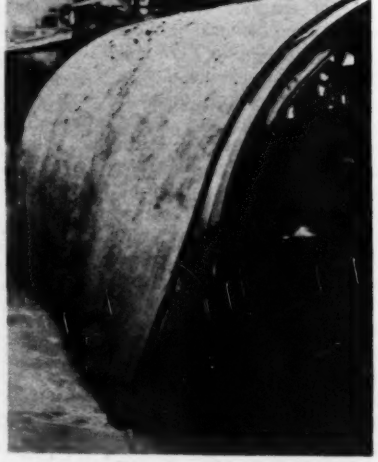

Perforated pipe spray for cooling steel plate on which mat is molded
Steel plate molding deck
Perforated pipe spray for cooling upper surface of mat
Asphalt mat
Friction rolls for launching cables
Roll of reinforcing
Reel of cable
Water level
Scow
Tension device for stretching wire mesh reinforcing

Fig. 3—Appearance of asphalt mat as it passes over launching apron into the water.



COST OF MATTRESS FOR REVETMENT
PER SQUARE OF 100 SQ. FT. OF BANK PROTECTION

Type	First Cost[1]	Effective Life in Years[2]	Annual Cost[3]
Improved 1932 design articulated-concrete mattress	$10.50[4]	25	$0.42
Improved 1930 New Orleans articulated-concrete mattress	17.82[5]	15	1.19
Original Vicksburg articulated-concrete mattress	10.70	10	1.07
Framed-willow mattress	17.21[6]	10	1.72
Slab-concrete mattress	10.38	8	1.30
Fascine-willow mattress	16.04	8	2.00
Woven-willow mattress	9.00[7]	4	2.25

[1]Based upon 1931 work except when otherwise noted.

[2]That period of time during which the mattress retains its integral continuous structure and is capable of resisting the *direct continuous* attack of the river. Based upon observations made during underwater survey. Life of most recent types estimated.

[3]Derived by dividing first cost by effective life. Maintenance is ignored. Maintenance will vary from 5 to 10 per cent per year for all types except the first two, on which it will be very much less.

[4]Estimated on basis of average 1931 experience with articulated-concrete mattress in Memphis District.

[5]Cost out of line because of small volume of widely scattered work. Large plant with 200,000-square capacity actually placed only 17,000 squares.

[6]Cost for 1930 work. Is out of line as compared with fascine-willow mattress because of small volume.

[7]Estimated on basis of experience with other willow mattress. Last mattress of this type was built in 1893.

Costs have been given somewhat circumstantially, as they point rather directly to the reason that has influenced the energetic study of the last two years of revetment economics. Action has been largely in two directions: (1) reduction in construction cost of the standard revetment types; and (2) search for substitute types or structures that will reduce investment and be more enduring. As a preliminary to new study a comprehensive survey was made of revetment in place. Incidentally, this survey of river revetment is the first ever conducted on a comprehensive scale.

Scope of revetment survey

The function of bank revetment on the Mississippi is to prevent subaqueous scour. Other objectives follow or are a coodinate element of scour prevention, but they are distinctly resultant objectives or elements. The scour area of the river bank, as considered in revetment protection, extends from the bank top into the river to the deep water of the lowest contours of the channel valley or thalweg of the river. At bankfull and flood stages all of this bank width, and at the lowest water stages a large part of it is subject to the current action of flowing water. Revetment then is underwater structure, and largely it is underwater construction. The purpose of the survey was, as indicated by these facts, the determination of: (1) the river influences affecting revetment structure and materials and the nature of the effects; (2) the revetment design and construction factor influencing durability; and (3) the life values of various types of revetment in service. Its scope included investigations of 538 sections of revetment at some 30 locations on both banks of the river for 980 miles from Cairo to New Orleans.

The examination was made between Aug. 7 and Nov. 13, 1931, by trained divers who traveled over each revetment along instrumentally located paths and reported by telephone to recorders above water the facts found as the examination proceeded. The exact position of the diver was known as he reported, and it was recorded with his remarks. All records were accurately plotted in comparison with the plots of the original position of the revetment when constructed. The record when completed was of enormous volume, and only salient findings are possible of mention here.

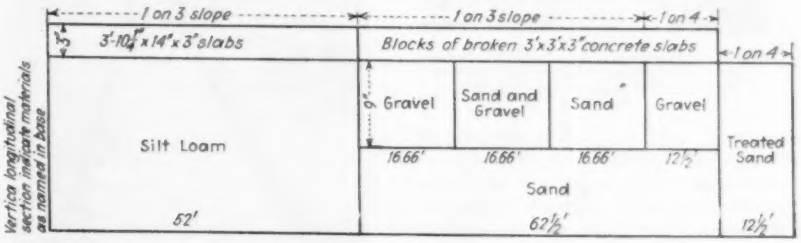


Fig. 4—Diagram of arrangement of flume beds and types of riprap for laboratory tests of riprap revetment.

Agencies affecting revetment

The action of the river on revetment as determined by the survey divides into effect on revetment material and effect on revetment structure. In the survey the condition of materials was determined both by bringing samples to the surface for examination and by ascertaining by underwater examination where and for what reason the functional integrity of the material had been destroyed. Condition of structure was ascertained by underwater determination of the position and structural integrity of the revetment mat. Slips, settlements, bends, punctures, breaks and disappeared parts were recorded, and the reasons for each, as far as possible, were determined. These records, interpreted with the collateral information given by soundings and continued observation of river and bank action, gave the data for defining the causes of failure.

The agencies affecting revetment material were found to be corrosion, abrasion and natural deterioration of quality. These effects were briefly as follows:

Corrosion—All types of revetment, both willow brush and concrete blocks or slabs, are bound together or articulated by steel strand (rope) or wires, which are exposed. It was found that corrosion of iron and steel not buried in silt in Mississippi River water is not materially slower than above water, and it is probably much faster. Embedded in silt, these metals corrode slowly. Galvanized strand of several wires—i.e., wire rope—rusts away completely in three to ten years. Commercial-wiped galvanized coatings are worthless as protection against corrosion. Large wire of equal breaking strength to cable, or strand composed of small wires have a longer life in water. Copper-covered wires show some i olated corrosion accounted for by earlier manufacturing defects now corrected; otherwise they are in good condition after six years of use. Silicon bronze wire after twenty years indicated virtually no corrosion. No evidence was found of any special electrolytic corrosion.

Abrasion—Abrasion equals corrosion as a destructive agency. It results from the rubbing together of the mattress parts and from the eroding action of sand and other materials carried into contact with the revetment structure by the current. The brush parts of willow revetment are particularly damaged by cutting, but this action also affects the metal parts at junctions and intersections. Abrasion of concrete parts of revetment was insignificant. Wire ties and the wood of willow mattress have been widely cut through and separated from the mat structure.

Change in Material—Concrete showed no material disintegration or loss of quality. Willow, however, showed a rapid loss in quality, changing within three years from a stringy resilient material capable of enduring repeated bending to a brittle dead wood breaking easily when bent. The rotting of the leaves and smaller twigs also loosened up the mat and increased the opportunities for corrosion and abrasion.

The factors responsible for the destruction of revetment considered as a structure were found to be scour, bank sinking, improperly prepared bank or emplacement and structural weakness. Scour of the bank material out through the covering mattress and undercutting

Fig. 5—Revetment of irregular blocks broken from 3x3-ft.x3-in. concrete slabs as laid on 1 on 3 bank ready for test.

scour at ends and outer edge are the chief destructive agencies acting on revetment. Such scour or bank sinking leaves unsupported edges, holes and hollows under the mat and often steepens the bank slope. The results are that the mat is dislocated by sliding, the undermined edges break off and settlement into holes and hollows pulls the mat apart. Scour through the mattress is chiefly of interest in its bearing on revetment construction. It results from seepage outward through the bank of groundwater, but more extensively from the suction by turbulent current of bank material through the interstices of the mat.

Both inherent structural weakness and unavoidable construction faults were found to have been the cause of many failures. Another considerable destructive cause has been insufficient bank preparation, particularly failure to remove snags, stumps and other protuberances under water.

Conclusions from survey

Based on the survey, the following conclusions were reached concerning the effectiveness of the several types of revetment:

1. Willow-fascine mattress — Both willows and steel wires and cable have a short life. The mattress fails in as short a time as six years and rarely lasts ten years under direct attack.

2. Willow-frame mattress—This construction is more impermeable and durable than fascine construction. It has little exposed metal, the willows are better compacted, and individual willows are nailed in place. The mattress fails under direct attack in about ten years.

3. Concrete-slab mattress—The slabs cannot be assembled in proper order under water. The mat is highly permeable even if assembly were possible. The cables rust away in about ten years, leaving a disordered array of ineffective slabs; the concrete slabs remain in good condition.

4. Articulated concrete-block mattress —This construction has gone through three stages of improvement. The original Vicksburg construction is unsatisfactory because of the short life of cables and wires; it falls apart under direct attack in about ten years and becomes an aggregation of unassembled blocks with the concrete still in good condition. In the New Orleans construction, developed from the original, longer life was sought by larger wires and stronger fastenings; the assumed life is more than fifteen years. A further improved construction in 1932 strengthened the mattress by the use of heavier blocks and metal connections and protected steel. The "expected effective life" is at least 25 years.

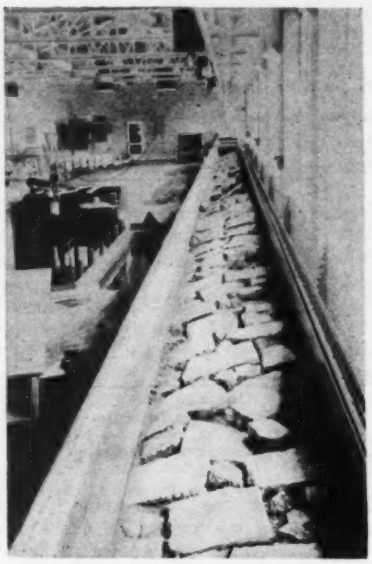


Fig. 6—Irregular broken-slab revetment after 4-hr. 55-in. test run, showing displacement and base erosion.

In general, the investigation disclosed that revetment has shown insufficient impermeability, flexibility and longitudinal and transverse tensile strength. In willow construction the deterioration of material was found to be general. As indicated, the latest articulated concrete-block revetment has been materially improved in all of these respects. In addition, exceptional experimentation has gone on in a search for new types of revetment or substitute structures for holding caving banks.

Submerged groin experiments

One of the first of the new devices experimented with was submerged groins, one set constructed on a stable bank in shallow water with light currents near New Madrid, Mo., and another on a caving bank in deep water with swift current near Happy Valley, Tenn. Two types of groin were built, one trench filled with rock and the other with a row of steel sheetpiles. The two types are indicated by Fig. 1. In all cases the groins were built normal to the bank, from bank top out to deep water, with their tops flush with the bank slope. They were spaced differently for observation from 50 to 300 ft. apart. The cost of construction at New Madrid was $19 per linear foot for pile and $22 per foot for rock groin. At Happy Valley the pile groin cost $26 a linear foot. At New Madrid the groins remained intact two years after construction, but those at Happy Valley were soon destroyed except for remnants that were afterwards removed.

Asphalt-mat experiments

The most promising field experiment with mat revetment has been the construction and laying of a wire-reinforced asphalt-mastic mat in New Orleans harbor. These experiments have continued about a year, beginning with tests of asphalt mixtures for flexibility, strength and low cost. The mixture selected consisted of 66 per cent sand, 22 per cent loess and 12 per cent of asphalt. This in sheets withstood repeated bendings without fracture.

The method of fabricating the mat is indicated by Fig. 2. As shown, the reinforcement (about 2x4-in. mesh with the strips laced together at the edges) is led from the reel over a steel molding deck. The procedure is similar with the mooring cables. With the steel in place, the asphalt mixture at about 250 deg. is deposited so as to embed the steel in the middle of the 2-in. slab. Hand methods were used in the experiment for placing and compacting the asphalt. With the slab finished it is cooled by water sprays to about 100 deg. and slid down the

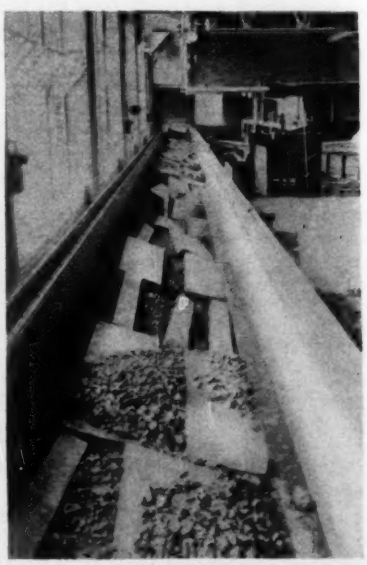


Fig. 7—Regular block revetment after 4-hr. 55-min. test run, showing displacement and base erosion.

apron of the molding deck, exposing a new area of steel for embedment. The process of molding and launching is continuous. As the mat reaches bottom and spreads out the molding scow is moved outshore.

The plant indicated and the procedure described were wholly experimental to determine the practicability of the new revetment. A working plant now being built consists of a group of barges used for the preparation and sinking of the mats. There will be a machinery-equipped barge where the asphalt will be mixed; the mattress barge on which the various sections of the mat will be constructed; the supply barges for the sand and the loess; a mooring barge equipped with cables to pull the whole plant upstream, and the string-out barges that keep the mattress barge from shifting position with the current.

The sand, loess and asphalt will be mixed on the mixer barge and will be sent in a screw conveyor to the mattress barge. The mattress barge will measure 260 by 50 ft., and each launch (or section) of the mattress will measure 218 by 30 ft.

The trial plant and operation have given sound encouragement that the new revetment is practicable and probably cheaper than concrete revetment. Its durability has yet to be determined. One fact in its favor is that none of the reinforcing or carrying cable steel is exposed, and there are no joints to permit scour of bank material out through the mat.

Flume tests of riprap revetment

To obtain indications of the behavior of covering and base material with riprap-type revetment, two sets of flume tests have been conducted at the U. S. Waterways Experiment Station at Vicksburg, Miss. One was a test of reduced-size molded tetrahedral blocks of concrete, and the other was an undistorted test of rectangular blocks and of irregular blocks produced by breaking concrete slabs. In both tests the blocks were set on bases of various materials.

Tetrahedral Block Test—The blocks were molded 10x5 in. to simulate 50-lb. blocks for actual construction. These blocks were deposited by dropping them 5 ft. on base material with a lateral or bank slope of 1 on 3. The bank materials were sand, gravel and clay placed in about 40-ft. sections with approach and exit sections of concrete-paved bank in a flume 165 ft. long and 3½ ft. wide, with a longitudinal slope of 0.001. The riprap material was subjected to velocities of 3 to 5 ft. at depths of 1.5 to 1.86 ft. The runs were five hours long, and twelve runs were made.

The series of runs indicated in general close nesting of the blocks and little movement down the bank slope. It was concluded that the tractive force required to move a 50-lb. block longitudinally was greater than that of any Mississippi River current. The covering remained stable on clay and gravel bank; the sand bed leached out between the blocks, allowing settlement. Using small blocks to seal voids proved less effective than gravel fill. The flume test gave no indication of possible bank undercutting at the edge of the covering.

Riprap Tests—The significant feature of these tests is that natural sizes and conditions were reproduced. The treatment was the same material, the banks were of the same slope, and the bank materials were those found in Mississippi banks. The flume test was therefore less a model test than a test of a section of full-size structure. The arrangement and materials for one set are indicated by Fig. 4. In the other set the blocks laid on a sand bank were subjected to exceptionally swift currents.

Briefly the revetments were: (*a*) mattress blocks with spacing gradually widening from ⅜ in. at one end to 14 in. at the other end; (*b*) broken blocks spaced from ¼ to 3 in.; (*c*) sand penetrated with sodium silicate and calcium chloride; (*d*) mattress blocks as in *a* spaced 2 to 2½ in. on a 1 on 3 sand slope. On *a*, *b*, and *c* the run was 4 hr. 55 min., with velocities from 1.56 to 4 ft.; on *d* the run was 44 min., with velocities varying from 6 to 7 ft. A summary of the results is as follows:

That reach of revetment where the spacing between blocks was ⅜ to 2¾ in. showed little erosion of the silt; considerable erosion occurred where the spacing was from 6 to 14 in. Revetment *b* failed to protect the bank against erosion, which was deepest on the sand base. Treated sand, section *c*, completely failed. Under velocities reaching 7 ft. *d* revetment failed in 44 min.

Knowledge of revetment behavior and of the various causes contributing to revetment deterioration has been materially extended by the surveys and experiments conducted in 1931 and 1932. The possibility has been demonstrated of producing from hot asphalt mix a continuous mat that can be molded and submerged at about the speed of assembling and launching articulated concrete-block mat. Tests of riprap revetment indicate a degree of stability that encourages actual trial of this type of protection, and plans are now being worked out for a tetrahedral concrete-block casting plant. Experimental bank paving with these blocks is contemplated. In the tentative plan the blocks will be graded in size from large blocks at deepest water to smaller blocks at the bank top. This arrangement is based on the principle that tractive force increases with depth of flow and requires the heaviest blocks in the deepest water

Ohms of Resistance Measure Concrete Curing

Tests of cement-spray method of curing concrete pavement introduce method of measuring curing efficiency by determining electrical resistance

By Searcy B. Slack
Decatur, Ga.

A NOVEL METHOD of sealing the surface of concrete pavement for curing has been proposed by F. A. Hipple, Atlanta, Ga., and tried out on a concrete-paving job under the direction of the state highway board of Georgia. The process consists essentially of spraying the surface of the concrete, just after finishing, with a thin layer of neat cement paste. The claim is that the fine particles of cement will be driven into the surface pores of the concrete and will serve both to close these openings and to harden the surface of the concrete. As the surface is sealed with this paste the moisture will be retained and cure the concrete, and the concrete will not in the future absorb moisture so readily.

The machinery for applying the cement spray consisted of a small air compressor and a mixing tank. In the mixing tank there was an agitator for keeping the cement paste of uniform consistency. About 45 lb. of air pressure was used to force the cement paste out of the mixing tank through the hose and nozzle. At the nozzle the cement paste was broken up into a fine spray by a stream of compressed air. The method of application, the mixing tank and air compressor are all shown in Fig. 1. The amount of cement used was one sack for each 100 sq. yd. of surface sprayed. Too heavy an application would cover the surface sand grains and give a slick surface, while too little would not properly cover the surface.

The concrete pavement on which the tests were carried out was the usual 9-6-9-in section with doweled longitudinal center joint and transverse expansion joints at 50-ft. intervals. The aggregates were quartz sand and crushed granite proportioned by weight for a mix approximating 1:2:3½ (1.45 bbl. cement per cubic yard of concrete). Curing on all of the project except the test sections was by covering the concrete just after finishing with wet burlap, which stayed in place about 24

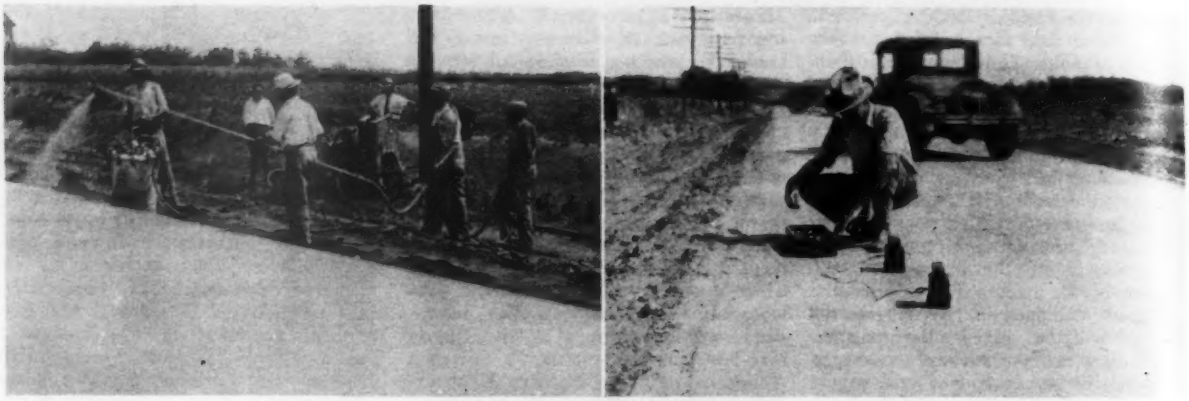


Fig. 1—Equipment for and method of spraying cement paste on concrete pavement (left) and apparatus for measuring electrical resistance of concrete (right).

hours. As soon as it was removed the pavement was covered with a 3-in. layer of earth and kept wet for ten days.

The sections on which the tests were carried out were all constructed during the first week in June, a very hot dry week. On Monday, Wednesday and Thursday the pavement was cured by earth and water in the usual way. At the beginning of each day's run during this week a section 100 ft. long was cured by covering with wet burlap for 24 hours with no further curing.

Four cores 6 in. in diameter were bored from each long section, and two cores were bored from the 100-ft. sections. All cores were bored when the concrete was 28 days old and were tested the same day they were bored. The averages of the results of the compression tests on these cores are as follows:

	Lb. Per Sq. In.
Cured by earth and water	3,388
Cured by cement spray	2,961
Cured by wet burlap 24 hours..	2,716

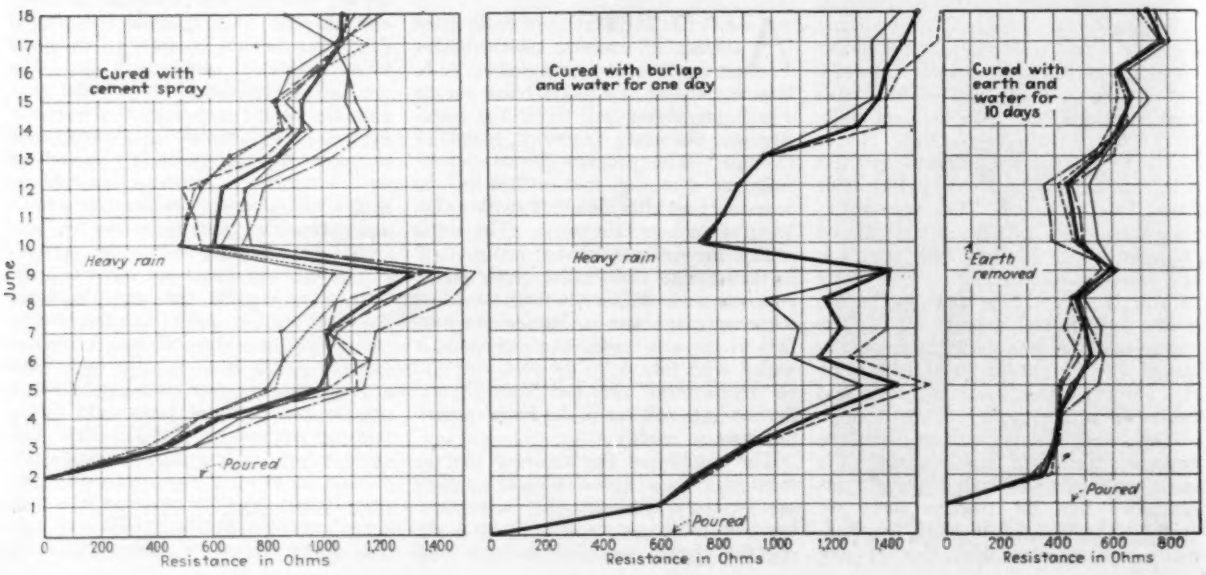
Another method of comparing results of the different methods of curing was used on this project. It is generally assumed that an effective cure holds the moisture in the concrete. The electrical resistance of concrete varies with the amount of moisture, the resistance being low for moist concrete and high for dry concrete. A measurement of the electrical resistance of the concrete would then give some measure of the moisture and hence of the curing.

The resistance measurements were made between points 3 ft. from the edge of the pavement and 3 ft. apart. Quite a number of points were used in each section, so that the average would be fairly representative. Two methods of getting contact with the concrete were tried: the first, by using electrodes made from strips of galvanized iron ¼ in. wide and embedded in the concrete 2 in. This method was not satisfactory, as the electrodes were hard to place uniformly and would be kicked loose by persons walking on the concrete. The second method of making contact was by two 4x4-in. wood blocks, one end of which had been covered with sheet copper and the sheet copper covered with layers of cheesecloth. The cheesecloth was dipped into a solution of salt water and placed on the surface of the pavement. The cloth would take up irregularities in the surface and through the salt water give good contact with the copper plate. Reasonably uniform results could be secured with contacts made in this way.

In measuring the resistance of the sections being cured with earth and water the pavement was uncovered for some distance on each side of the points between which the measurements were to be made. The surface of the concrete was allowed to dry out until no surface moisture showed, before the measurements were taken.

The results of these resistance measurements on three of the sections cured by different methods are shown in the curves Fig. 2. These curves are typical of the results on other sections. It is interesting to note the effect of the heavy rain that fell during the afternoon and night of June 9. These results indicate that a resistance method of measuring moisture in concrete, or curing, has interesting possibilities.

Fig. 2 — Electrical-resistance curves obtained on concrete pavement cured by three methods, one being a spray coat of cement paste.



Record Floods in Colorado, Wyoming and Arizona Formulized

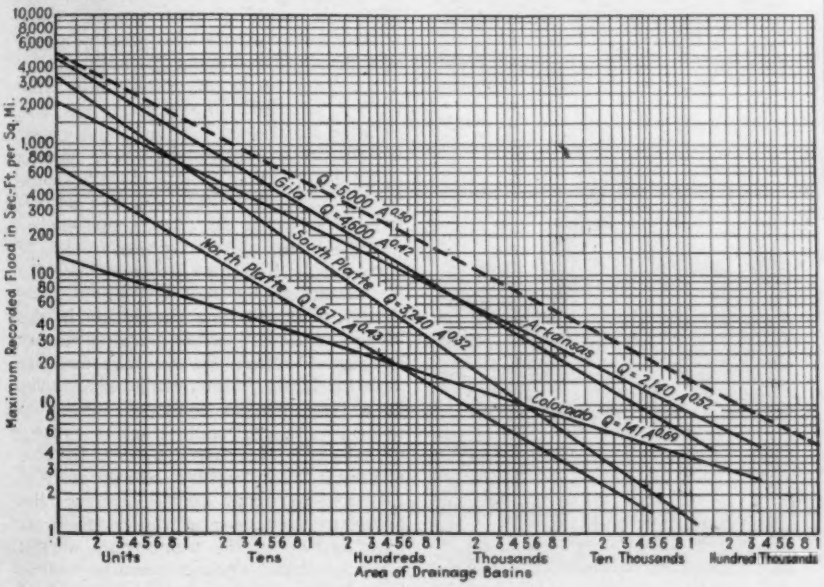
By Paul V. Hodges
Hydrographic Engineer, United States Indian Service

THE FLOODFLOW characteristics of the principal streams in Colorado, Wyoming and Arizona have been analyzed from existing flood data; and the relation of peak discharge to the various basin areas is presented in the accompanying formulas. Flood prediction for each stream is based primarily on the recorded floods from the same drainage basin.

Flood discharges are usually represented by a formula of the type $Q=CA^n$ showing the relation of the size of flood to the area of the drainage basin, with Q the discharge in second-feet, C a coefficient, A the drainage area in square miles and n an exponent. The formula is open to criticism on the grounds that there are other factors of importance besides the drainage area, which contribute to the characteristics of floodflow. However, this type of formula has comparative value in evaluating the flood-producing characteristics of different streams, and it has been used for the present study as a basis of comparison only.

For a more comprehensive analysis such factors as the shape and configuration of the basin slope of the stream and storm characteristics of the locality should always be taken into consideration.

Comparison of floods in the different basins shows that certain areas are more subject to excessive storms than others. Arizona and the eastern foothills of the Rocky Mountains are especially subject to cloudbursts, and small areas tributary to the Uncompahgre River in the southwestern part of Colorado have intense storms.



Comparison of flood formulas of principal streams in Colorado, Wyoming and Arizona. These formulas and others for lesser rivers and areas are presented in the table.

Records in the eastern half of the United States indicate that excessive storms cover an area about 15 miles in diameter, or about 180 square miles. In the arid West, rainfall stations are usually so far apart that the area covered by excessive storms cannot be definitely determined. Local storms in this region, however, cover relatively small areas, but they occur often and there may be several local areas having intense precipitation at the same time.

Drainage areas applicable to the floodflow formulas in the table.

The flood formulas for the various basins in each state are given in the table, and the corresponding areas are indicated on the sketch map. As a means of comparison of characteristics, the principal basins have been compared by plotting the flood formulas on logarithmic paper. In addition, the modified Myers formula, $Q=5{,}000\sqrt{A}$ for 50 per cent maximum floods (*Transactions*, Am. Soc. C. E., Vol. 89, p. 995) is shown to give a relation between the formulas derived and a general formula.

Future floods

The prediction of future floods must of necessity be based on floods of record, and the most reliable basis consists of the records in the same basin as that of the river under investigation. The formulas presented give some basis for comparison of the probable maximum floods to be expected for the different basins. Each stream is a separate problem, requiring separate analysis and, while general formulas are useful as comparisons, they are of limited value in determining future floods. Formulas, when used, should be derived from areas of similar characteristics and applied only to such areas.

This article is based on the flood record data of the U. S. Geological Survey.

FLOOD FORMULAS FOR VARIOUS RIVER BASINS

State	Map Reference	Stream	Area	Flood Formula
Colorado	1	Arkansas	Basin	$Q = 2{,}140\ A^{0.53}$
		Arkansas	Less than 500 sq.mi. subject to cloudburst	$Q = 4{,}780\ A^{0.53}$
	2	Colorado	Basin	$Q = 141\ A^{0.69}$
		Uncompahgre	Small areas subject to cloudburst	$Q = 1{,}720\ A^{0.53}$
	3	South Platte	Basin	$Q = 3{,}240\ A^{0.32}$
Wyoming	4	Big Horn, Powder and Cheyenne	Basin	$Q = 759\ A^{0.58}$
	5	Green	Basin	$Q = 891\ A^{0.48}$
	6	North Platte	Basin	$Q = 677\ A^{0.43}$
		North Platte	Less than 200 sq.mi. subject to cloudburst	$Q = 1{,}120\ A^{0.43}$
Arizona	7	Gila	Basin	$Q = 4{,}600\ A^{0.42}$
	8	Colorado and Little Colorado	Basin	$Q = 6{,}170\ A^{0.50}$

Preserve the Landmarks!

Great losses now incurred by the destruction of survey monuments and benchmarks can be reduced by care in setting permanent marks for all surveys, thoroughness in tying survey systems together and popular education on the importance of landmarks

By William Bowie
Chief, Division of Geodesy, U. S. Coast and Geodetic Survey, Washington, D. C.

A SIGNIFICANT STATEMENT appeared in the report of the land-surveying committee of the Corporation of British Columbia Land Surveyors a few months ago: "It is a strange matter that every material thing except a surveyor's landmark seems to belong to someone. The accumulated cost of establishing landmarks in the past must be terrific, and yet no one seems to claim ownership to them or to take any interest in their upkeep or preservation once they are set."

No doubt the losses resulting from the destruction of landmarks, not only in British Columbia but in other countries, has been very great. In the United States millions of dollars have been spent in establishing the section corners of our public land surveys alone; yet how many of the monuments designating the section corners have been lost through the carelessness of man or the action of natural forces? For example, in highway construction section corners have been dug up in many cases, and often the witness trees and other marks that would enable the engineer to re-establish the section corners have also been destroyed.

Carelessness and treasure-hunting

The Coast and Geodetic Survey has, during its 117 years of existence, established tens of thousands of triangulation stations and level benchmarks over our land area and along our coasts. Many of these marks have been destroyed, some by erosion, others by industrial development, still others by carelessness or thoughtlessness on the part of engineers who were building roads or carrying along other engineering work, and many others have been destroyed by treasure hunters. In nearly every section of the country there are traditions that treasure has been buried, some of it by pirates and some by travelers who are supposed to have buried their treasures when they were attacked by Indians, and an obscure surveying monument in the form of a block of concrete or stone has often led the finder to think that treasure was buried under it. It was dug up and the survey station was lost in consequence.

The Coast and Geodetic Survey and other federal mapping agencies are now using inscribed metal tablets for their survey monuments in order that any one seeing them will know just what they represent; it is surprising to note the greater respect that is paid these monuments by the construction engineers than was given the obscure monuments that carry no inscription. Some of the states have passed laws protecting federal survey monuments from wilful destruction, and though it cannot be said that these laws are rigorously enforced, yet they do furnish a certain degree of protection to the monuments. Far better, however, would be an enlightened public opinion regarding the value of the monuments.

Figs. 1 and 2—Many history-making boundary lines are marked by substantial monuments. The Mason & Dixon line monument near Delta, Pa. (above), marked with the Lord Baltimore arms, is an example of how a famous line is fixed by permanent marks. Even more secure is Boundary Monument No. 1 of the 49th parallel boundary line between the United States and Canada on Point Roberts, Strait of Georgia (below). Its location is latitude 49°00′ 08.03″, longitude 123°05′ 16.95″.

It is interesting to note that landmarks have been the subject of legal action for thousands of years. Back in the time of Moses, 1500 B.C., the laws protected survey monuments. Moses commanded the Israelites to set up great stones and cover them with plaster as they passed over the Jordan into the promised land. On this plaster should be written all the words of the laws in order that the people might know what these laws were. One of the laws was to the effect, "Cursed be he that removeth his neighbor's landmark, and all the people shall say Amen." Job, in speaking of the wrongdoings of his people, listed the removal of landmarks as one of the reprehensible acts they committed. Solomon in his Proverbs said, "Remove not the ancient landmarks which thy fathers have set," and again he said, "Remove not the old marks."

These Biblical quotations deal with the boundary marks of private property. There is nothing that causes more trouble to the owner of a piece of property than to have doubt raised as to the exact location of his land. He may wish to sell the land and give clear title to it, or he may wish to raise money on it. In each case he must be sure that the exact locations of his boundaries are known. His boundaries will be known if he has placed substantial monuments at their turns, setting the monuments in such a way that they are not likely to be disturbed by man or nature. He should, whenever practicable, tie in the boundary corners of his property to any horizontal control system that may lie near his land. The control survey may be a triangulation connected with the federal horizontal control net, or it may be a local survey based upon plane coordinates. In either case, if the spherical or plane coordinates of his boundary corners are known, then as long as two of the monuments of the control survey system remain undisturbed he can relocate his boundary corners with the assurance that they are correct.

The surveyor's omission

If the loss and destruction of surveying monuments of all classes in the country have cost enormous sums of money, the failure to monument survey work has cost probably more. It is surprising how much surveying is done in connection with engineering work of all kinds for which no survey monuments were left. The number of miles of traverse and leveling run in connection with surveying work of all classes, especially in highway location, cannot be estimated, but the sum is enormous, and in most cases survey monuments are not placed as the work is done. The result is that a survey made for a special purpose cannot be utilized for other purposes. It is probable that a line of good leveling run through a county for a special purpose might be used, if prop-

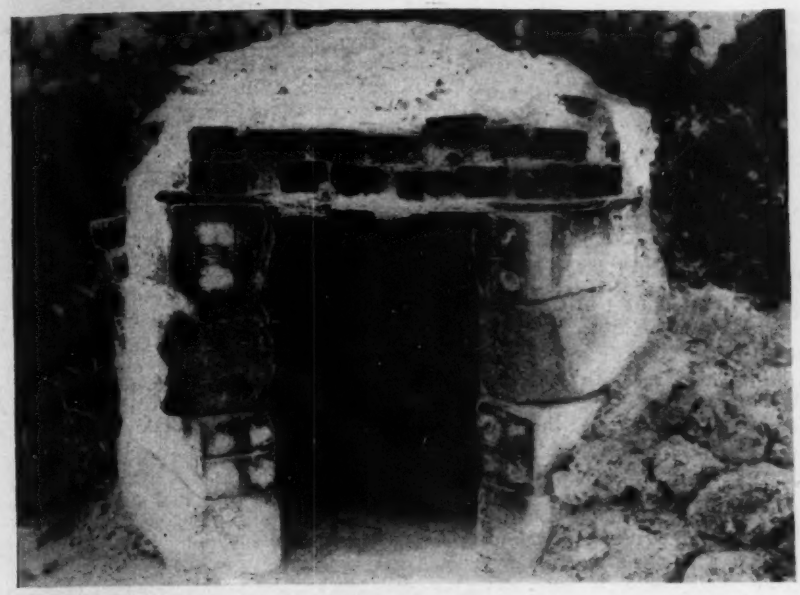


Fig. 3—Work of the landmark vandal—a bake-oven in Florida built of stones that once marked triangulation stations in that vicinity.

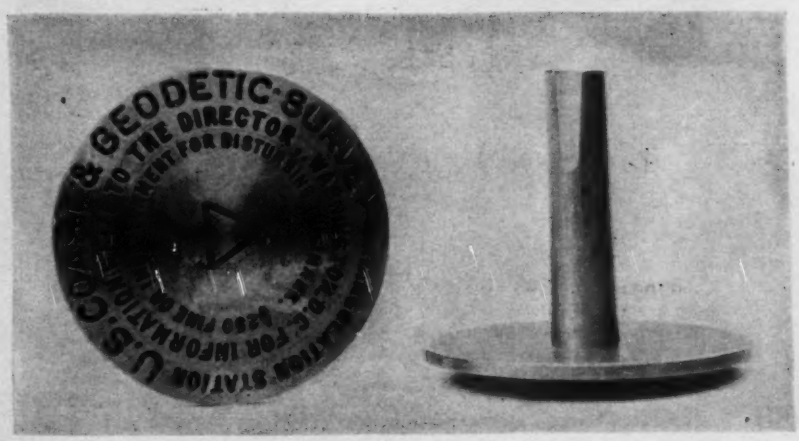
erly monumented, in hundreds of ways for scores of years. But if the first line of levels is not monumented, another must be run when a second special project is executed.

There is no use crying over what it lost, but let us all see to it that some of the waste that has been suffered in the past does not occur in the future. Long-range plans should be made, and the result of the work that is done this year should be perpetuated for the years to come.

Local landmarks often non-existent

The Coast and Geodetic Survey and the U. S. Geological Survey, in carrying on their national work, write to officials of states, counties and cities and tell of the routes along which they will extend triangulation and topographic mapping. They request data for existing benchmarks, triangulation stations and traverse stations established by officials of the political subdivisions of the country in order that they may be tied into the national nets. Such connections serve to expand the control survey nets of the country at no additional cost, and enable local control surveys to be used in the topographic mapping, thus making it unnecessary to duplicate the existing control surveys. But in the majority of cases the state and local officials reply that they do not have any survey data of the kind requested!

Figs. 4 and 5—Simple construction of a well-marked triangulation station. At right, a concrete monument cast in the ground, with upper part shaped by a metal form. Below, bronze disk marker set in top of concrete.



This condition, obviously, is most unfortunate, for the lack of monuments on the original surveys means that the lines must be rerun, which costs money. The funds used for these resurveys could be used in making more topographic maps or control surveys if the original surveys had been monumented.

However, conditions are getting better. A few decades ago the property owner was loath to give permission to the engineer engaged on surveys to enter his property for the purpose of setting a benchmark or a triangulation station; today he is glad to have such a mark on his land, for he realizes that the triangulation or traverse station can be used for perpetuating the boundary corners of his land and that the benchmark right at hand makes it possible for him to carry on more effectively work that may involve draining or irrigating his land. It may be that the engineers engaged upon public surveying and mapping were to blame for the destruction of many survey monuments; they may not have told the public just what they were doing and why. It is a very simple matter to carry the message to the public and let them know how the public surveys over their area will benefit them.

Set permanent marks

Let us pay heed to the exhortations of the patriarchs of thousands of years ago not to remove the landmarks, and at the same time let us do what they probably did—that is, establish landmarks. The landmarks should not only be put in but they should be of such a character that they cannot easily be disturbed. A small amount of concrete will make a mark, and to establish such a mark will cost an insignificant amount as compared with the cost of doing the survey work all over again. Elevations, positions, distances and bearings are the fundamental data needed in many projects. The results of control and boundary surveys furnish these essentials, but only if the lines are monumented and the monuments respected and protected.

Minnesota Highway System Expanded

The Minnesota legislature has added 4,400 miles of roads to the trunk highway system of the state, increasing the system to a total of 11,400 miles. The new routes added to the system include those passing through the three first-class cities, Duluth, Minneapolis and St. Paul. Heretofore, highway funds had not been available for use on the routes through these cities.

Castlewood Dam Failure Floods Denver

Old structure of combined rockfill and masonry type is overtopped and half of it washed out by cloudburst flood, releasing flow of 16,000 sec.-ft. into the 10,000-sec.-ft. Cherry Creek channel running through Denver 35 miles north of the dam

ON THE MORNING of Aug. 3 the suspense of a generation of residents and business men along Cherry Creek in Denver was relieved and the predictions of some engineers and others were realized when the Castlewood Dam situated on Cherry Creek 35 miles southeast of Denver failed from the pressure of floodwaters resulting from a cloudburst above the dam. The entire east half of the dam, including the spillway, went out, leaving the west half standing. Between 7,000 and 8,000 acre-ft. of water were involved in the flood. Fortunately, only two lives were lost; and while considerable damage resulted to farm lands, city and railroad property and to perishable goods stored in Denver warehouses, this damage was not so great as some had predicted it would be if the Castlewood Dam should ever fail.

M. C. Hinderlider, state engineer of Colorado, in reply to a telegraphic inquiry as to probable cause states: "Castlewood Dam failure, following 43 years of service, was due to unprecedented runoff, estimated at from five to six thousand sec.-ft., topping dam to 1-ft. depth over its entire length. Failure doubtless due to erosion of lower toe of the spillway located over the top of the dam or to displacement of rubble masonry in spillway section, causing collapse of loose-rock fill composing the main body of the dam Consider no spillway of reasonable capacity would have been effective."

Dam of unusual design

The Castlewood Dam (*Engineering News*, Feb. 9, 1899, and *Engineering Record*, Dec. 24, 1898) was of unique design, consisting of a rockfill placed downstream from a masonry wall and paved with large masonry blocks. The upstream wall was about 4 ft. thick with a batter of 1:10. The downstream paving was carried up in steps on a slope of about 1:1. The wall and paving were joined at the top to form a coping about 8 ft. wide and 4 ft. high. The length of the dam was about 600 ft. along the crest, and its maximum height was 70 ft. above the streambed and about 92 ft. above the foundation, which consisted of a clay boulder formation. According to the drawings now available, the rockfill rested directly on the boulder formation while the masonry walls up and downstream were carried from 5 to 20 ft. below this level. An overflow spillway 100 ft. long and 4 ft. deep was provided near the center of the dam and was supplemented by a 40-ft. bypass channel on the west side of the dam. A 6x7½-ft. outlet well was provided in the center of the dam. Eight 12-in. inlet pipes at different elevations admitted water to the well, from which it was conveyed by a 36-in. concrete conduit to the creek below the dam. The surface area of the reservoir created by the dam was about 200 acres, and the capacity was 3,500 acre-ft. Since the construction of the dam, the water impounded in the reservoir has been used to irrigate lands in the Cherry Creek Valley.

Cherry Creek lies in the cloudburst region of Colorado, comprising the area along the foot of the Front Range and along the west side of the Plains, and the area in the Arkansas River basin between the mountains and the Plains region. The drainage area of Cherry Creek is about 450 square miles, all of which is above the city of Denver, the junction of Cherry Creek and the South Platte River being near the Union Station in Denver. About 175 square miles of the drainage is above the Castlewood Dam site. Several floods have occurred in Cherry Creek, the most notable of which were those of May 19 and 20, 1864, July 14, 1912, and the present one of Aug. 3, 1933.

Flood damage

It is reported that about a 46-ft. depth of water was in storage in the reservoir on Aug. 2 when heavy rains of cloudburst proportions caused floods in Lake Gulch, Antelope, east Cherry Creek and main Cherry Creek above the Castlewood Dam. At 1 a.m., Aug. 3, the water was running over the top of the dam, and at about 1:20 a.m. the dam failed. It is estimated that the water was about 1 ft. deep on the crest of the dam when failure occurred and that about 5,000 sec.-ft. were passing through the spillway and over the dam at that time. The flood resulting from the failure of the dam reached the outskirts of Denver between 5 and 6 a.m. and arrived at the state gaging station on the South Platte River at 19th St., about four blocks below the mouth of Cherry Creek, at 6:10 a.m. The peak of the flood passed that point at 7:55 a.m. with an estimated discharge of 16,000 sec.-ft. Below Denver the flood peak flattened rapidly so that very little damage was done to irrigation structures.

Since the improved channel of Cherry

Fig. 1—Castlewood Dam after noodwaters had swept the east half of the structure, including the spillway. The break reveals the rockfill core behind the stepped masonry downstream wall. The outlet well is shown at the break. At the far end of the dam is the bypass channel built to supplement the spillway.

Creek through Denver has a capacity of only about 10,000 sec.-ft., the flood submerged an area several blocks wide on either side of the creek. Property of the city of Denver sustained damages in an amount estimated between $200,000 and $250,000. This consisted of the complete destruction of the bridges across Cherry Creek on Colorado Boulevard, Stout and Champa Sts.; partial damage to the Logan St. Bridge; the destruction of a foot suspension bridge across Cherry Creek; the flooding of the Sunken Gardens in the vicinity of Cherry Creek; and the undermining of some portions of the retaining walls that form the sides of the Cherry Creek channel through the city. Several leaks developed in water mains crossing the creek, but these were not serious and were quickly repaired. A telegraphic report from Walter H. Wheeler, consulting engineer, Minneapolis, who designed the flat-slab concrete bridges over Cherry Creek at Downing, Curtis, Market, Larimer, Colfax and Logan Sts., states that some hand-rail damage occurred at Downing, Logan and Market Sts., but that otherwise the bridges were undamaged.

Considerable damage resulted from the deposition of mud and silt in the Union Terminal station and the complete flooding of the subway. Most of the railroad yards in the vicinity of the station were flooded. Several bents of the Denver and Salt Lake Railroad bridge across the Platte River were lost. Two bridges of the Burlington Railroad were partly destroyed. A large amount of material was lost from the yards of the Colorado and Southern Railroad.

The flood inundated a considerable portion of the warehouse district of Denver, which lies along the lower portion of Cherry Creek. A large amount of damage resulted from the submerging of perishable goods. It is believed that this damage will amount to at least $100,000. No buildings except a few small residences and other small buildings were damaged to any great extent.

While many of the grounds in the fashionable country club residential district were flooded, no great damage was done in this section. It is reported that about $5,000 damage resulted to the golf course and grounds of the country club.

Fig. 2—Cherry Creek channel in Denver did not have great enough capacity to carry the flood. View shows the warehouse district under water and the Cherry Creek junction with the South Platte River.

Damage to lands in the Cherry Creek Valley above Denver resulted from sand deposition and destruction of crops and fences. Many head of live stock were lost.

Dam repaired in 1898

The construction of the dam was begun in December, 1889, and was completed in November, 1890. It was built for the Denver Land and Water Co. from plans prepared by A. M. Welles, its chief engineer. In Wegman's "The

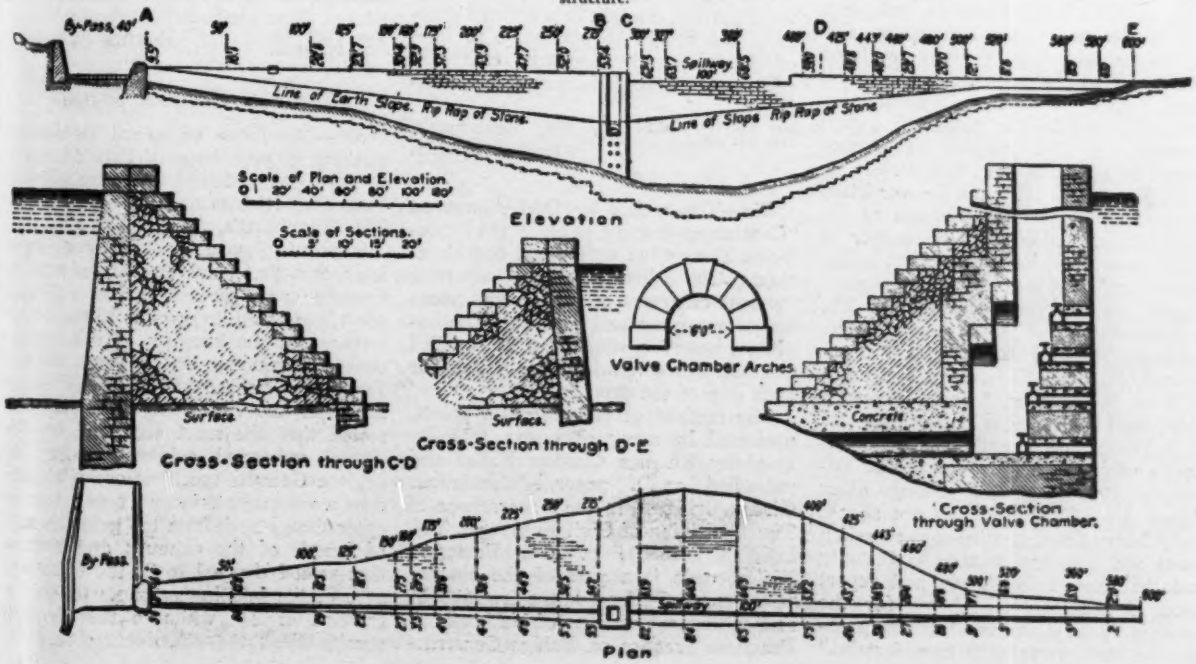


Fig. 3—Design details of the Castlewood rockfill dam in Colorado, built in 1889. After partial failure in 1897, earth embankment, shown in plan and elevation views, was placed against upstream face, mainly to stop leakage under masonry structure.

Design and Construction of Dams," sixth edition, p. 277, it is stated: "The plans of the Castlewood Dam have been severely criticized, especially the fact that the inner face-wall, where only 4 ft. thick, overhangs the loose rockfill, its center of gravity falling outside its base. Eight years of service have, however, not yet shown any disadvantage resulting from this feature."

Shortly after completion of the dam, settlement occurred which caused the opening of cracks in the rubble masonry face, causing leaks. Repairs were made in 1898 under the supervision of the state engineer and consisted of the construction on the upstream side of the dam of an earth embankment 8 ft. wide on the crest, with a slope of 3:1 faced with 12 in. of riprap. The outlet works were also extended by constructing a 40-in. wood-stave pipe incased in a 12-in. shell of reinforced concrete from the valve chamber to the upstream toe of the embankment.

Flood control studied

From 1907 to 1911 and again in 1917 much work was done to improve the channel of Cherry Creek through Denver. Land was acquired, and an 80-ft. channel was designed and constructed, with reinforced-concrete retaining walls on each side. A boulevard was built along one side of the channel. This work cost about $1,000,000, about one-half going for the purchase of land and one-half for construction work.

Some engineers familiar with the situation believe that the channel is not sufficiently large to carry floods that may be expected to occur in Cherry Creek and that additional means for flood protection should be provided. Many plans, in fact, have been proposed. One involves the construction of a detention reservoir or reservoirs in the Cherry Creek basin above Denver on the assumption that floods may be expected with very large peaks but producing a relatively small total volume of water. If such floodwaters could be held in storage for a short period, the flood menace to Denver would be removed, they believe.

In November, 1931, the Denver Planning Commission recommended to the city administration that the matter of flood prevention be investigated. The mayor appointed a group of engineers to investigate and report back to him. The group consisted of A. K. Vickery, city engineer, Robert Follansbee, district engineer of the U. S. Geological Survey, William B. Freeman, Arthur Ridgeway, chief engineer, D.&R.G.W. Railway, and R. J. Tipton, special engineer, state engineer's office. This committee, in December, 1931, reported among other things the following: "We are agreed that Cherry Creek continues to remain a great and constant menace to the city and that a complete study and report upon a flood-control scheme for the creek is the first and most urgent problem for such commission to undertake."

Letters to the Editor

Continuous-Beam Design

Sir—Any method that will facilitate and expedite the solution of statically indeterminate problems deserves the serious consideration of those interested in the structural analysis of restraint and continuous beams, particularly relative to reinforced-concrete construction. In this connection the republication of Dr. W. Ritter's solution involving Culmann's graphical analysis, as described by Odd Abert in the article "Continuous-Beam Design by the Fixed-Point Theory" (*ENR*, June 29, 1933, p. 842) is timely and well presented.

It may be of interest to state that the writer has presented Dr. Ritter's method in full in the *Transactions*, Am.Soc.C.E., Vol. 90, 1927, p. 142, wherein a solution of several practical problems and the proof of Dr. Ritter's general method may be found. Furthermore, Dr. Ritter's method has also been illustrated and discussed in the *American Architect*, May, 1922, p. 445. As a matter of fact, the whole process is really one of graphical differentiation and integration.

It is noted that Mr. Albert compensates for the change in moments of inertia by adjusting the reverse third-point lines in accordance with the ratios of the spans and moments of inertia, although the proof is not given. Another method of adjustment for variable moments of inertia is to vary the span lengths in accordance with the stiffness ratio of I/L, wherein the adjusted lengths of the spans vary inversely as the stiffness ratios.

Ritter's graphical solution in place of the tedious standard analytical methods presents an efficient and easy solution to what otherwise might be a difficult problem, and should be conducive to improvements and economy, especially relative to reinforced-concrete construction.

Los Angeles, Calif., July 11, 1933. R. McC. BEANFIELD, Consulting Engineer.

* * *

Sir—The article by Odd Albert on "Continuous-Beam Design by the Fixed-Point Theory" is well worth consideration. The directions for drawing moment diagrams of loaded continuous beams are clear and concise. The engineer, however, should not be satisfied until he has determined the reason for each step of the process.

The method of characteristic points, preferred by many engineers, was devised by Thomas Claxton Fidler and published as a paper, "Continuous Girder Bridges" in the *Proceedings* of The Institution of Civil Engineers, Vol. LXXIV (1883), p. 196. Professor Fidler made it a part of the chapter "Continuous Girders" in the first (1887) and subsequent editions of his book "A Practical Treatise on Bridge Construction." It is a matter of regret that textbooks almost invariably ignore the method. An elementary knowledge, aside from Fidler's book, may be obtained from the section on continuous beams in Andrews "The Theory and Design of Structures," London, fifth edition, 1932. The 37-page monograph "Characteristic Points," by Ernest Hinkly Salmon, published in 1927 by The Institution of Civil Engineers as No. 46 of "Selected Engineering Papers," treats the subject with great thoroughness. Dr. Salmon makes far wider application of the method than that suggested by Professor Fidler. One case is finding the bending-moment diagram for the stiffeners in a partly filled water tank; another is the treatment of bracket loads on columns.

A noteworthy paper, "Moments in Restrained and Continuous Beams by the Method of Conjugate Points," by L. H. Nishkian and D. B. Steinman, is published in *Transactions*, Am.Soc.C.E., Vol. 90, June, 1927, pp. 1-206. The conjugate-point method is similar to the characteristic-point method, although not identical with it, as stated by some that took part in the discussion of the papers. A wide range of application of the method is brought out; in fact, the discussion with the original paper is a valuable symposium.

The "best" method to follow in continuous-beam design involves the personal equation. To quote one who took part in the discussion (R. G. Doerfling): "The best method, of course, for any one person and also the shortest and safest method for him, is the one he understands best and feels sure about, be it graphical, analytical or a combination of these methods."

New York City, July 17, 1933. ROBINS FLEMING.

Concrete Column Spirals

Sir—My letter on spiral reinforced columns in your issue of Feb. 23, 1933, p. 257, was followed by three others: Professor Richart's, April 13, p. 477; Mr. Hirschthal's, April 27, p. 536; and Mr. Scott's, June 1, p. 719. From these it appears that numerous circles are interested in the question raised and that the views of the experts as to the effectiveness of the hooping of reinforced-concrete columns differ very widely. Professor Richart has correctly characterized the conflicting opinions and stated that the most economically designed unhooped columns under the present German specifications will produce a very unsatisfactory type. Among other things he fears a too great change of length of the columns and asserts that with l/d equal to 20 the effectiveness of the hooping amounts to only a fraction of 2.0, while in the German specifications the effectiveness is eval-

mated at 2.5. American tests have shown the value of 2.0 for $l/d = 7.5$. (European tests have in part yielded values far beyond this.) Permit me to consider briefly these objections and to set forth some details of the German specifications that should allay Professor Richart's fears.

First of all, as regards the ratio $l/d =$ 20, the German specifications stipulate for this a buckling coefficient of 1.7. In a 24-in. square column with a 20-in. core diameter, if $l/d = 20$ then $l =$ 400 in., and the length referred to the length of a side is 400/24 or 16.67, for which the buckling coefficient in columns with simple loop reinforcement (without hooping) amounts to only 1.083. This example should show clearly that the decrease of effectiveness of the hooping in the German specifications, mentioned by Professor Richart, is allowed for by the buckling coefficient, which is introduced already from a ratio of $l/d = 13$ upwards and rises very rapidly with an increasing ratio. In the example cited with equal outside dimensions the load of a simply reinforced column must be increased only by 8.3 per cent, while on the other hand the load of a spiral reinforced column must be increased by 70 per cent. From this it follows that in the case of ratios greater than $l/d = 13$ there is little inducement to use spirals, since their advantage as against columns with simple loop reinforcement — namely, saving of space—is lost through the different treatment of the buckling increases.

The use of a hooping (F_s) having three times the value of the longitudinal reinforcement (F_e) has been common practice in Germany for decades. The relation $F_s = 3F_e$ has been adopted in the official specifications since 1916, and no unfavorable incidents of any kind have been reported.

The German specifications place a limit on an excessive increase of the hooping by providing that the calculated load part of the hooping may not be greater than the calculated part of the full concrete cross-section and the longitudinal reinforcement together. As the allowable calculated stress amounts to one-third of the prism strength, then the greatest stress of the concrete under the calculated assumed full load, if the hooping is still supposed to be wholly ineffective, would be two-thirds of the prism strength equals one half of the cube compressive strength. According to Dr. Schreyer's investigations ("Elasticity and Strength of Concrete on the Basis of Cube Tests." *Beton u. Eisen*, 1933, vol. 3, p. 42) the extension of the concrete at this stage amounts to 0.5 per cent. In a height between floors of 20 ft., which can be considered not more than normal, the shortening in the most unfavorable case would not be even ½ in.

The change in the 1932 German specifications compared with those for 1925 consists, as regards the columns, in allowing higher steel and concrete stresses, when the calculated assumed concrete strengths are guaranteed by a standing control on the one hand, and on the other hand in allowing reinforcement percentages up to 8 per cent of the core cross-section for the longitudinal reinforcement, while formerly 3 per cent of the full concrete cross-section was the upper limit. As a consequence of the above-mentioned limitation, the increase of the hooping is of course much smaller. The relation $F_s = 3F_e$ soon ceases with increasing longitudinal reinforcement and falls, at the upper limit of 8 per cent to about 1.

Darmstadt, Germany. June 14, 1933. K. HAJNAL-KÓNYI.

Making the Highway Pay Its Cost

Sir—The article by Bernard E. Gray, highway engineer, Asphalt Institute, entitled "Making the Highway Pay Its Cost," in your issue of May 11, 1933, p. 590, is an interesting economic discussion based on license fees and gas taxes paid by motor-vehicle owners. The author makes it particularly clear that the total cost of motor-vehicle operation is not considered.

In discussing economics of highway transportation it is natural to make comparison with economics of other transportation agencies, and with railways particularly because they have so much in common with highways.

After reading Mr. Gray's article a multitude of questions dealing directly with the cost of rail transportation entered my mind, such as: How do the railroads consider their costs of transportation, and how do railway engineers determine when and where grade reductions are warranted, and when and where new rails are needed?

The railroad engineer will tell you that he figures a grade reduction is warranted when the extra freight carried by the same locomotive over the grade will pay for cutting down the hill. Or, in case the same loads are hauled, when the saving in coal will pay for the work.

It seems inescapable that the economics of highways are the same as those of railroads in many ways. The cost of using the transport medium is an integral part of the transportation system. To disregard the cost of gas, oil, tires, etc., in highway transport is analogous to disregarding the wear and tear on locomotives and the expenditures for fuel in rail transport.

Then, too, the cost of using highways is really a tremendous amount of money that annually exceeds the cost of the highways themselves. About 15 billion gallons of gasoline was used on our highways last year. At an average cost of 10c. a gallon for the gasoline this represents an expenditure of about 1½ billion dollars. Then we have the oil bill, the tire bill and the car-repair bill, all representing more millions of dollars directly applied to highway transportation.

From another angle we can determine the money represented in using the highways, the item disregarded in large part in the article referred to above. It is recognized that *total* cost of using roads varies between 5½c. per vehicle-mile on concrete to 7½c. per vehicle-mile on earth. Using 7,000 miles as the annual travel of each car and a total automobile registration of about 20,000,000, we have a total annual transport bill paid by the automobile owner that is somewhere between $7,700,000,000, the very minimum, and $10,500,000,000, a probable maximum. The article in question analyzes about 10 per cent of this total, or $800,000,000, as the gas and vehicle taxes paid by the vehicle owner.

Such stupendous expenditures involved in the use of highways must have an influence on the economics of highway transport. To ignore ways of reducing the major portion of these expenditures through improvements in highways is similar to the sudden decision of the railroad man that he no longer needs to consider his coal bill in determining the cost of operation.

It seems obvious, therefore, that the cost of using a transportation agency is an integral part of the cost of that agency and that highways are no different in this respect than railroads, waterways or air lines. The inclusion of *all* vehicle-operation costs is recognized definitely by the Highway Research Board of the National Research Council and many state highway departments. The use of *all* these costs in an economic analysis of highways leads to far different conclusions regarding the types of improvements that will pay for themselves.

For example, such an analysis was reported by T. H. Cutler, chief highway engineer of Missouri, before the fall meeting of the American Society of Civil Engineers at St. Louis in 1930. Mr. Cutler showed that for Missouri conditions all roads carrying an average of 210 or more vehicles per day should be paved. Further, that at least 12,000 miles of Missouri roads should be paved to meet economic needs. The Michigan state highway department exhibit at the Highway and Building Congress in Detroit in January, 1933, portrayed the economics of highway building and showed that all roads carrying 190 or more vehicles per day should be paved.

These two examples also illustrate another important point recognized by the highway fraternity. The highway engineer, in building roads, is the agent of the people. As their agent, he is responsible for all transportation costs and therefore must consider not only the immediate highway expenditures but also the influence of these expenditures on the cost of using the roads.

Chicago, Ill., May 18, 1933. MILES D. CATTON, Highway Engineer, Portland Cement Assoc.

Engineering News-Record

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Constant Vigilance Needed

FAILURE of Castlewood Dam on Cherry Creek above Denver lays new emphasis upon the need for continuing supervision of dams, especially those known to be none too stable. The fact that a dam is amply safe and sound at the time it is put into service relieves no one of responsibility if it fails in after years, for age has a deteriorating effect on the best of structures and unforseen weaknesses may develop at any time. Unfortunately, though many of our states now have laws providing for periodic inspection of dams by the state engineers, parsimonious legislatures do not provide funds sufficient to carry forward the work on a scale sufficient to insure adequate and continuing inspection of a character calculated to reveal the slow development of serious defects. Failure of Castlewood Dam was due primarily to scour at the toe caused by overtopping resulting from a sudden flood greater than any in its 50-year life. Under some circumstances it might well be said that this long record established the adequacy of the spillway against all but unforeseen storms. But in a region subject to rainstorms of "cloudburst" intensity, adequate study doubtless would have shown that the spillway lacked the factor of safety required for a dam poised above a populous city.

Landmarks

A SPECIAL APPEAL to engineers to render dual service in the preservation of landmarks is made in this issue by William Bowie, of the U. S. Coast and Geodetic Survey. Dr. Bowie asks not only that they help in educating the public to the need for preservation of existing landmarks, but also that they make a direct contribution to the cause by seeing that the marks that they set or that are set under their direction be well set, of a permanent nature and properly referenced. As he so clearly point out, many surveys have been made that would be of invaluable aid to government engineers in new work or in resurvey work had their value not been destroyed by failure to set permanent marks. No engineer would be guilty of wilful destruction of an existing landmark, yet he does what amounts to the same thing if he set a wooden stake that will rot in a few years when the mark is of such value to others as to make a permanent marker desirable.

Survey Funds

THE PROFESSION generally will find satisfaction in the allotment by the Public Works Administration of $2,600,000 to the Coast & Geodetic Survey and $2,400,000 to the Geological Survey for surveying and mapping. Engineers more than any other class of citizen are conscious of the inadequacy of the basic survey data in this country. Vast areas are still unsurveyed, and in other regions the basic surveys have not been revised for nearly half a century. The result is much loss of both time and money in preliminary studies. Recently engineers, either as individuals or through organizations like the American Society of Civil Engineers and American Engineering Council, have made a concerted effort to check drastic curtailment of the work. This allotment of $5,000,000 is evidence that the administration is convinced of the soundness of their argument. Added satisfaction may be found in the fact that the proposed work will permit the re-employment of men who have already been laid off and will provide new employment for many other engineers.

About Wood Borers

RECORDS of wood structures destroyed by insects offer few examples more curious than that of the St. Paul, Minn., factory building described in this issue. Here the destruction of wood-pile tops, which left column footings standing over clusters of very smooth round holes, gives engineers something to think about both as to the use of wood piles and the reasons why nothing more happened than a moderate settlement of the building. The claim used to be made that a Howe truss would stand up on principle even if the timber had rotted, and somewhat the same quality must have been incorporated in the fifty-year-old St. Paul factory. Seriously, however, engineers have a problem that cannot be slighted in guarding timber structures against the wood-eating insects and sea worms that abound in many localities under various conditions. In general, engineering knowledge is more certain concerning marine wood-borers than it is of the work of those that live in the air. Investigations so far made by the Forest Products Laboratory of the activities of termites have given contradictory results on the effects of different preservatives in different locations. The studies being made are, however, a promise that engineers are being put on the road to information about a problem of which a better understanding is required.

Two Great Developments

IN A WEEK rendered sensational by the number of constructive steps taken toward industrial recovery—foremost among them the tardy promulgation of instructions for public-works applications—two events only indirectly related to construction stand out prominently. One of them is the agreement to settle labor disputes by arbitration instead of by strikes and lockouts. The other is the Interstate Commerce Commission's denial of requests for a general reduction of rail freight rates. The significance of the labor truce needs no emphasis, in view of the grave character of the numerous labor differences that were springing up and the prospect that they might paralyze much of the Recovery Administration's accomplishment. The rail freight decision, however, is no less vital to economic rehabilitation. Any further curtailment of railway revenues would almost certainly have threatened many lines with bankruptcy, and what this would have done to our none-too-securely-repaired financial structure may readily be imagined. The commission's decision, there-

fore is a matter for general congratulation. It helps to strengthen assurance that every form of business and industry is steadily moving toward solvency.

Small-Scale Models

THE INCREASING practical importance of small-scale model tests again is emphasized in the tests on models of the Boulder Dam spillways reported by E. W. Lane in this issue. These tests resulted in significant changes in the spillways as built from those shown in the original designs because the tests definitely established the effect of factors not susceptible to exact mathematical determination. In earlier tests on models many questioned the value of the results obtained, especially where the scale was small, on the ground that the prototype would not necessarily perform in a manner similar to the model. This fear has been found to be largely unfounded when proper allowances have been made in interpreting the results. In the course of the Boulder Dam tests the dependence of small-scale models was further confirmed. There, because of the magnitude of the forces to be dealt with in the spillways, tests were made at scales up to as large as 1:20. When the latter were checked against those made at scales of 1:60 and 1:100, remarkable similarity was noted.

Professional Codes

CHANGES in matters relating to codes for engineers and architects have been rapid, almost kaleidoscopic, during the past two weeks. Whereas less than a month ago the possibility of professional codes under the Recovery Act still was a subject for academic discussion, now an architects' code prepared by the American Institute of Architects has been presented to the policy committee of the Construction League of the United States as a supplementary code under the general code for the construction industry, and a code for engineers has been drafted in tentative form by the American Society of Civil Engineers. The latter also will be submitted to the League and both will be in the hands of the NRA just as soon as the League's policy committee can adjust any possible conflicts with other codes for the construction industry.

The explanation of this rapid change in the professional-code situation is not hard to find. It lies in the fact that, though loosely held together, all parts of the construction industry are essential to all others, and as a consequence no one major element can operate successfully under a code without making it compulsory for the other elements to do so. This condition, as we noted briefly last week, lays new emphasis upon the unity of the construction industry.

Recognition of the bearing of this essential unity upon the question of codes for the construction industry about two weeks ago brought order out of chaos and also brought many architects and engineers who had believed that codes for professional groups were neither necessary nor desirable under the Recovery Act to the opinion that such codes are necessary even though not desirable from a purely selfish point of view.

From the start of the discussion of codes for industry as a whole it was obvious that general contractors would be required to act under a code of fair practice and competition, but when such a code came to be worked out in detail it became apparent that contractors alone could not control such practices as bid-peddling, and that the cooperation of architects and engineers was essential. Under the system of self-regulation required under the Recovery Act no branch of industry can set up regulations for another branch in its code; if such regulation is necessary, it must be set up within the other branch. The conclusion then became obvious that at least some practices of architects and engineers must be controlled in a code. But if a few practices had to be controlled, it became apparent that *all* competitive practices must be. The outcome was a decision to submit complete codes covering all competitive practices.

Other developments that had an important bearing on the decision of the two leading professional organizations in the construction field to submit codes of practice to the NRA were, first, the fact that many local engineering organizations and groups of specialists were preparing codes and were submitting them to the NRA, and second, the decision of the deputy administrator that because of the impossibility of dealing separately with the multiplicity of codes affecting the construction industry that were pouring in upon him he would not consider any code for that industry unless it had cleared through the policy committee of the Construction League, that organization having offered its services in clearing up the situation. This decision, as noted in our issue of July 27, resulted in the rejection of the code of the Associated General Contractors as a basic code and the submission by the Construction League of a basic code for the whole construction industry. With the A.G.C. code set up as a secondary code under that basic code, a logical development called for parallel codes from architects, engineers, subcontractors and materials producers and distributors. If endless confusion in the engineering field were to be avoided, it was necessary therefore that the American Society of Civil Engineers, the controlling engineering group in the construction field, take the lead in the preparation of an engineers' code.

These circumstances should be taken into consideration by all those who within the next few weeks will be asked to pass on the engineers' code as prepared for presentation to the Construction League and, ultimately, to the NRA. It is not a perfect document, as those who have labored over it well know, but it should not be rejected for that reason. No amount of work upon it would make it entirely satisfactory to all concerned. In the present instance, time is an essential element. When the blanket code for the construction industry—now before the NRA—is approved by the President, it becomes effective over all elements in the industry, *including* engineers who design and build any structure. That code is too general to be satisfactory to engineers and does not deal specifically with many practices that for years have been sources of trouble within the profession.

What is needed at the moment, therefore, is a code that will meet present requirements and cover the principal issues between members of the profession. This the code now in preparation does. Detail points can be ironed out subsequently. Unless sources of serious difficulty are found in the code it should be approved, so that it can be considered concurrently with the architects' code. Non-essential details must not be permitted to obstruct early action to make an engineers' code effective as an integral part of construction industry control.

NEWS OF THE WEEK

Water Power Study By Federal Body Launched by N. R. A.

Washington Correspondence

PRESIDENT ROOSEVELT'S instructions to the Federal Power Commission to prepare a comprehensive national plan for the development of water power and the transmission of electricity is regarded as a major step towards establishing public ownership and operation on a firm footing throughout the country. With an allotment of $400,000 from the public works fund, the commission will begin its investigation immediately. Such an investigation was ordered by the Senate last session. In the preparation of the national plan the commission will organize its work on a regional basis so that local projects proposed to the Public Works Administration may receive prompt consideration.

The federal commission will cooperate with state public service commissions, the Tennessee Valley Authority and other agencies and experts.

Coordination lacking

The Public Works Administration has been advised by the power commission that up to the present time the development of water power resources, both public and private, has been carried on without reference to any comprehensive or integrated national plan. As a result, there have been, according to the commission, many instances in which the full economic and social value of the nation's resources has not been secured. In other cases, the commission states that there has been a lack of proper coordination between projects developed in the same region. Power projects also have been developed in some areas without proper regard for the available market, according to the commission. Summing up, the commission asserts that the failure to develop a national program has resulted in great economic loss.

National planning is represented by the Commission as giving effect to the purposes of Congress which in the Federal Water Power Act of 1920 authorized the Commission to investigate "the water power industry and its relation to other industries and to interstate or foreign commerce and concerning the location, capacity, development costs and relation to markets of power sites and whether the power from government dams can be advantageously used by the United States for its public purposes," etc.

The commission's investigation will be directed to determining the facts regarding the cost of transmitting and distributing electricity to consumers under varying conditions, as this is recognized as of great importance not only in ascertaining the economic feasibility of power projects, but also the distribution cost element properly to be included in rates to consumers of electricity.

Announcement of the investigation followed shortly after the appointment of Frank R. McNinch, of North Carolina, as chairman of the Federal Power Commission, to succeed George Otis Smith.

Beside the allotment of $400,000 to the power commission, the allotments made by the Public Works Board this week included $20,000 to provide working quarters for the Bureau of Reclamation in the Custom House at Denver, $8,000 for repairing a break in mains laid by the Jefferson Parish Water District No. 2 in the Intercoastal Canal, La., and $6,000 for repairs to the coping of the roof of the White House.

Casper-Alcova project opposed

Allotment of $22,700,000 for Casper-Alcova, a federal reclamation project in Wyoming, has been ratified formally by the board but, due to a protest from the state of Colorado, the board now has to decide whether or not to reconsider this allotment. Because of the priority that is established by the beneficial use of water in western states, Colorado for years has insisted that an agreement protecting its rights to a share of the water of the North Platte should precede approval of the Casper-Alcova project in Wyoming. Colorado will take the case to the U. S. Supreme Court if the Public Works Board carries the project forward.

Included in the $22,700,000 allotted for the work on the North Platte River is construction of the Seminoe storage reservoir above the Pathfinder reservoir.

A hearing on Colorado's protest was held by the Public Works Board on Aug. 8. The allotment originally had been made by the board when informed by Senator Kendrick that Governor Johnson had assured him that Colorado would not insist upon a prior agreement.

Local work still held up

Secretary Ickes stated Aug. 8 that the Public Works Administration still has to expand considerably its staff in Washington before state and local projects forwarded by the state advisory boards can be given expeditious attention. The appointment of state engineers is still hanging fire.

Mississippi levee building

No allotment has been made by the Public Works Board to date for carrying on revetment work and levee building on the Mississippi. Following receipt of a protest from Senator Harrison of Mississippi, against the interruption of work done by the army engineers with hired labor, Henry M. Waite, deputy administrator, now is taking this matter up with the Budget Bureau.

President Roosevelt's announcement that, in line with the NRA program, an adjustment would be made in government contracts where possible, does not extend, in Secretary Ickes' opinion, to contracts let by municipalities on projects financed by the Reconstruction Finance Corporation. Secretary Ickes previously had announced that the Public Works Administration could not grant any relief to contractors on the New Orleans Public Belt Railroad Bridge nor the Los Angeles Aqueduct.

Code for Engineers in Tentative Form Drafted at New York

ACTING under the spur of the Administration's desire to have as many codes as possible in hand by Sept. 1, the informal conference on a code for engineers that met in New York on Aug. 2, as noted in our issue last week, continued its deliberations over the greater part of two days under the chairmanship of Carlton Proctor, chairman of the American Society of Civil Engineers' code committee, in order that Mr. Proctor and the officers of the American Society of Civil Engineers might have the benefit of the views of the organizations there represented in the preparation of a draft of a code for submission to NRA as a supplemental code under the general code for the construction industry. Following the conference, changes in the tentative draft proposed during the conference were incorporated in a new draft that is now before the directors of the society for their consideration. The revised draft includes a schedule of minimum fees for professional services, prepared by a subcommittee under the chairmanship of Harrison P. Eddy.

Organizations represented

In addition to the representatives of the American Society of Civil Engineers who took part in the conference, the following were present: J. H. Griffin, representin the A.A.E.; Duff Abrams, Am. Conc. Inst.; Philip W. Henry, Am. Inst. of Consulting Engineers; H. H. Henline, A.I.E.E.; Henry C. Meyer, Jr., A.S.M.E.; P. S. Wilson and A. V. Ruggles, A.W.W.A.; J. A. Wright, Assn. of Consulting Chemists and Chemical Engineers; John C. Eadie, N. Y. Association of Consulting Engineers; D. B. Steinman, Arthur Sheridan, E. A. Kingsley and John W. Pickworth, N. Y. State Society of Professional Engineers; and H. V. Spurr, Structural Engineers Society of New York.

Architects and engineers agree

In our report of the opening session of this conference (ENR, Aug. 3, 1933, p. 146), it was stated that points upon which architects and engineers could not agree at the code discussion in Washington were referred to the code committee of the Construction League for a decision. This statement was in error. The architects and engineers settled all differences over their respective codes before completing their deliberations. The points referred to the code committee were concerned with the conflict between the codes for these two professions and that for the general contractors. Since the news item of last week was written, the code for architects has been submitted to the NRA. It provides for segregation of engineering fees for foundation, structural, mechanical and other engineering services.

PWA Will Deal With Cities Direct, Ickes Tells Governor Ely

The Public Works Administration through Administrator Ickes on Aug. 2 rejected unconditionally a request by Governor Ely of Massachusetts that it agree not to deal with municipalities in that state unless a board to be created by the by the Governor approve the proposed projects. Mr. Ickes informed the Governor that the Public Works Administration would consider solely on its merits propositions submitted to it without consideration of what the state might desire.

Governor Ely in reply has stated that cooperation in this regard between the state and the Public Works Administration is desirable in Massachusetts in order to protect the financial standing of the local communities.

Construction Code Work Centers on Supplementary Codes

Although tentatively submitted to the NRA, the basic code for the entire construction industry has been undergoing refinement and revision in terminology during the past week. The code committee of the Construction League has also been actively engaged in coordinating a number of supplementary codes with the basic code. It is hoped that when the basic construction code is in acceptable shape to be submitted for final approval that it can be accompanied by a considerable number of supplementary codes. By this procedure it is hoped to expedite the operations of the construction industry under the National Industrial Recovery Act. At the time of going to press no date had been set for hearings on the construction code.

Local Engineers To Be Employed on Columbia River Dam Work

The Denver office of the U. S. Bureau of Reclamation has announced that except for a few engineers in administrative capacities, the engineers employed by the bureau in the preliminary surveys for the design of the dam at Grand Coulee site on the Columbia River will be residents of the state of Washington. Last week the public works administration alloted $63,000,000 for the construction of a dam about one-third of the height required for the complete Columbia Basin combined power and irrigation project. The present development is to be for power alone. The federal government will grant 30 per cent of the cost of the labor and will loan the remaining 70 per cent to the state of Washington.

The preliminary work now being done by the Bureau of Reclamation under contract with the Columbia Basin Commission will take about six months. Construction of the dam and the power plant will require about five years. In addition to the investigation and design work being carried on by the bureau, Secretary Ickes has announced that $600,000 has been allocated to the construction of a railroad to the site and $600,000 for highways. Work on these elements of the project will be begun immediately.

The Bureau of Reclamation is establishing field headquarters at Almira, Wash., about 22 miles from the dam site.

Arthur Powell Davis, Boulder Dam Consultant, Dies

Arthur Powell Davis, who was named several weeks ago as consulting engineer on Boulder Dam, died in Oakland, Calif., on Aug. 7, following an illness of several months. He was 73 years old. Mr. Davis had served the federal government for many years, but had been out of its service for about a decade. About a year ago he returned from Russian, where he supervised the building of a hydroelectric system for the Soviet Government. He was credited with first envisioning Boulder

Arthur Powell Davis

Dam, and during his twenty-one years as an engineer in the Department of the Interior worked consistently for its construction.

Mr. Davis was born at Decatur, Ill., in 1861. He graduated from George Washington University and entered the federal service in 1882 as a topographer for the United States Geological Survey. Six years later he was transferred to the Irrigation Survey and was engaged in selecting and surveying reservoir sites in the Southwest.

With the passage of the Reclamation Act in 1902, he was made principal assistant engineer of the Reclamation Service, becoming chief enginer five years later and continuing in that capacity until 1914.

In that year the American Red Cross chose him as one of a group of engineers to investigate Chinese floods problems. Shortly after the board's report was completed in 1913, Mr. Davis became director of the Reclamation Service, a post he held until 1923. In his years of service Mr. Davis saw the construction of the Roosevelt Dam, Elephant Dam and Shoshone Dam.

In 1908 he was appointed one of a board of experts to report on Canal possibilities on the Isthmus of Panama. The board recommended the lock type of canal. The Kinkaid Act of 1920 directed Mr. Davis to report on the lower Colorado River, with special attention to flood protection of the Imperial Valley. It was his report on this work that recommended construction of a structure corresponding to Boulder Dam.

On last July 10, after being out of the Reclamation Service a decade, he was again chosen a consulting engineer in the new Bureau of Reclamation.

Chief Highway Engineer Named by Kentucky

In order to make its state highway department organization conform to the requirement of the federal aid law, Kentucky has named a chief engineer after having been without one for more than a year. W. O. Snyder, who has been with the Department for 12 years, has been named chief engineer.

Silicosis Suits on W. Va. Tunnel Settled Out of Court

All of the damage suits claiming contraction of silicosis filed by workmen against Rinehart & Dennis Co., contractor on the 3-mile power tunnel driven for the New-Kanawha Power Co. at Hawks Nest, W. Va., have been settled out of court. One of a multitude of suits came to trial in the Fayette County Court and ended in a jury disagreement after dragging on for nearly six weeks (*ENR* May 11, 1933, p. 603). At the time of settlement more than 300 releases were signed by those who had filed suits. Though details of the settlements were not made public, it is understood that the aggregate amount involved is considerable. The trial created much interest in the construction world, being the first case where a contractor was involved in claims for silicosis damage.

New Appointments Made To Public Works Organization

Additional appointments by President Roosevelt to complete the State Advisory Boards and the Regional Advisors of the Public Works Administration have been announced through Harold L. Ickes, Public Works Administrator.

George W. Lane, Jr., of Lewiston, Me., was appointed Regional Advisor for the First Region. Ralph L. Cooper, of Belfast, Me., was unable to serve.

Daniel C. Burns, of Denver, was appointed as a member of the Colorado State Advisory Board. Morrison Shafroth of Denver, announced he was unable to serve.

John E. Sullivan, of Somersworth, N. H., was appointed as a member of the New Hampshire State Advisory Board. Stanton Owen, of Laconia, died after his appointment.

Otto W. Deluse, of Indianapolis, was appointed a member of the Indiana State Advisory Board.

James E. Lawrence, of Lincoln, was appointed a member of the Nebraska State Advisory Board. James G. Maher, of Lincoln, was unable to serve.

Miguel A. Otero, Sr., of Santa Fe, was appointed a member of the New Mexico State Advisory Board. Felipe Sanchez y Baca, of Tucumcari, was unable to serve.

B. F. Moomaw, of Roanoke, was appointed a member of the Virginia State Advisory Board. Henry G. Shirley, of Richmond, was unable to serve.

Robert B. Preston, of Portsmouth, was appointed a member of the Virginia State Advisory Board. Richard Crane, of Westover, was unable to serve.

Walter J. Burke, of New Iberia, was appointed a fourth member of the Louisiana State Advisory Board.

Malcolm Pirnie Named NRA Assistant for Construction

Malcolm Pirnie, consulting engineer of New York, N. Y., has been named as assistant to Deputy Administrator Malcolm Muir of the NRA on construction and building materials. Mr. Pirnie also is acting as the coordinator between the NRA and the Public Works Administration. Mr. Pirnie was very active in the work of the National Committee for Trade Recovery, an organization that for more than a year fostered the enactment of federal public works legislation. He is also president of the American Water Works Association.

No Federal Funds Available for Wage Rise on New Orleans Bridge

That no money from the public works fund will be made available for the payment of wage increases on the New Orleans Bridge project has been announced by Harold L. Ickes, Public Works Administrator. The public works administration holds that to establish a precedent by allowing a grant of funds to make possible wage increases, it would mean that similar action would have to be taken on scores of other projects and would amount to admission on the part of the government that it would be responsible for losses on contracts of all kinds where action of the government has caused an increase in costs.

PWA Lists Information Required with Loan Applications

INFORMATION required by the Public Works Administration to be filed with applications for federal loans to public bodies under the public works act has been published as circular No. 2, available from the government printing office at Washington. The information required relates to the project itself, finances and legal considerations.

The text of circular No. 2 is as follows:

Information required with applications for loans to states, counties, municipalities and other public bodies

I. GENERAL INSTRUCTIONS.

Four (4) complete copies of the application and the accompanying papers and drawings should be submitted to the State Advisory Board, Federal Emergency Administration of Public Works, whose office is situated in the State in which the project is located. If the project is located in two or more States the application and the other papers should be submitted to the State Advisory Board in the State in which the principal office of the applicant is located.

All exhibits attached to the application should be given a convenient reference number.

The estimated costs required to be given should be based on the labor requirements and other special conditions set forth in the National Industrial Recovery Act, particularly section 206.

A study of the application may disclose the need for additional information. In order to expedite action on the application such information should be furnished promptly on request.

The application should state substantially the information called for below and in the order called for.

II. SUMMARIZED GENERAL INFORMATION.

(a) Full corporate name of applicant and the date of its organization.

(b) Name, title, and address of the official representative or representatives with whom correspondence should be conducted.

(c) Name and address of the applicant's attorney.

(d) Name and address of the applicant's consulting engineer or architect.

(e) The applicant's population in 1910, 1920, and 1930 (give brief explanation of any abnormal increase or decrease in population).

III. THE LOAN REQUESTED.

(a) Amount.

(b) Security (kind of obligation offered) i.e.—general obligation bonds, revenue bonds, etc.

(c) Period of the loan and maturities (annual amounts if serial maturities).

(d) Preferred method of application of grant if allowed (see Circular No. 1, page 14).

(e) Schedule showing dates when funds will be needed and amounts.

IV. THE PROJECT.

(a) General description with drawings sufficient for a thorough understanding of the project; and report of consulting or other engineer or architect if available.

(b) Estimate of the cost of the project subdivided into principal sections or items, in sufficient detail to permit checking, showing separately the following subtotals:

(1) Preliminary expenses;

(2) Cost of land, rights of way, and easements;

(3) Construction cost, subdivided into principal structures or classes of work, and showing (i) labor and (ii) material cost for each subdivision;

(4) Engineering charges;

(5) Legal, administration, and other overhead charges;

(6) Interest during construction;

(7) Miscellaneous costs in reasonable detail.

CASTLEWOOD DAM ON CHERRY CREEK FAILS, FLOODING DENVER 35 MILES BELOW

Overtopping of the Castlewood Dam on Cherry Creek, 35 miles southeast of Denver, as a result of a cloudburst upstream was followed by undermining of the toe of the dam and the washing out of half of the structure. The dam, a combined masonry and rock-fill structure, was about 600 ft. long at the crest and about 70 ft. high. It was built in 1889. The impounded waters were used for irrigation in Cherry Creek Valley. Release of the water by the failure of the dam caused a flow of about 16,000 sec.-ft. which could not be carried by the 10,000-sec.-ft. channel of Cherry Creek in the city of Denver. The warehouse district of the city was flooded, several bridges were washed out and considerable other damage caussed to both public and private property. A more complete report of the dam failure and flood appears on page 174 of this issue.

(c) Statement of the economic and social value of the project.

(d) Statement as to whether or not project is part of a larger plan or long-range program of development, and if so, give description of other parts of plan or program.

(e) Statement as to whether or not the applicant has a city or regional planning board, whether this project has been submitted to such board, and the views of such board, if any, in regard to it. (Report of planning board should be attached, if available.)

(f) Statement as to whether the community is part of a metropolitan district and, if so, whether the project is coordinated with the plans for metropolitan development.

(g) Statement as to all endorsements that are known to have been made by civic groups and other similar bodies in respect of the project or the application, together with names and addresses of such endorsers.

(h) Statement as to all objections that are known to have been made in respect of the project or the application, together with the names and addresses of such objectors; if no such objections have been made, that fact should be stated.

(i) Statement as to whether or not the project will compete with any similar enterprise.

(j) Statement as to status of drawings and specifications; when construction can be commenced; and when project will be completed.

(k) Statement of estimated average number of men to be employed each month, directly on the project, giving the number of months.

V. Revenue and Expenses. (Information to be submitted where loan, in whole or in part, is to be secured by pledge of revenues derived from the project.)

(a) If project is an extension of an existing system, give statement with appropriate subdivisions showing for each of the past three fiscal years:

(1) Operating income;

(2) Operating and other costs.

Statement should also show for each such year, the extent of the business of the system, as indicated by number of consumers or users, quantity of production or sales or other similar data.

(b) Give estimate with supporting data of annual operating income and of operating and other costs reasonably to be anticipated from the project if a new system, or from the existing system as extended or improved by the project, for the life of the loan. Give estimate showing for each year the extent of the business of the system as completed or as enlarged and extended, based upon the estimated number of consumers or users, quantity of production or sale or other similar data.

(c) A statement showing application of estimated income and dispostion of surplus for each year during the life of the loan.

(d) Discuss any unusual conditions affecting present or estimated future operating income and expenses.

(e) Give schedule of rates now in force and proposed rates after completion of the project, stating whether such rates are subject to approval by other public authority.

VI. Financial Data.

(a) Statement of assessed valuation of taxable property (showing real and personal property separately) for each of the last 3 fiscal years and ratio which such assessed valuation bears to actual value.

(b) Statement of gross funded debt at date of application, giving purposes for which each item of debt was incurred, classified as follows, and showing amount outstanding, interest rate, and maturities for each item:

(1) General obligation bonds,

(2) Revenue bonds,

(3) Assessment bonds, and

(4) Other kinds of funded obligations.

(c) Statement of sinking funds, showing amount applicable to each item of debt, and details of sinking-fund investments.

(d) Statement of gross floating debt at date of application, showing separately amount borrowed in anticipation of taxes and amount borrowed in anticipation of creation of funded debt, and showing interest rate and date of maturity for each item.

(e) Statement of gross debt of all overlapping taxing or assessment districts (i.e. school, paving, or other districts wholly or largely within the territorial limits of the applicants).

(f) Statement that all outstanding financial obligations of applicant and overlapping districts have been included in (b), (d), and (e) above.

(g) Statement of any defaults or postponements in payment of the principal of or interest on any outstanding obligations at any time, with explanation if any defaults or postponements.

(h) Statement of present total tax rate per $1,000 of assessed valuation, showing separately rate for State, county, city, school and any other overlapping taxing districts.

(i) Statement showing separately for each of the last 3 fiscal years amount of taxes levied (stating whether of total taxes, or of taxes levied for municipal purposes only), amount uncollected at delinquent date, and amount now uncollected.

(j) Statement of dates on which taxes are levied, on which they become payable, and on which they become delinquent.

(k) Statement of penalties and delinquent taxes.

(l) Evidence that the ordinary current expenses are within the prudently estimated ordinary revenues, or that action is in process or in good faith assured designed to bring this about. Attach last complete financial statement, and a statement showing, in reasonable detail, receipts and expenditures for next preceding 4 years, separating, as far as possible, ordinary receipts and expenditures from extraordinary items, such as capital improvements and creation and payment of debts.

(m) Brief general description of community, including information regarding railroads and main highways by which served, chief industries or commercial activities, products of surrounding country, and any other items of interest.

VII. Legal Data. A legal memorandum signed by counsel for the applicant discussing in detail all legal questions arising in connection with the project and the incurring of the proposed indebtedness should be attached to the application. Among other points the memorandum should cover:

(a) Reference to any general, special or local laws and State constitutional provisions pursuant to which the applicant was organized.

(b) If the applicant has a special charter, furnish a certified copy together with a certificate by proper officer to the effect that such copy is a true and complete copy of the present charter including all amendments to date.

(c) Give constitutional or statutory debt limitation. Give constitutional or statutory tax limitation and state whether such limitation includes debt service.

(d) State deductions which are allowed in ascertaining net debt stating separately amount of present gross and net indebtedness and character and amount of indebtedness for which deduction is claimed.

(e) Give reference to provisions of constitution and of general, special, or local laws empowering the applicant

(1) To construct the proposed project; and

(2) To issue the proposed obligations or otherwise finance the project in the manner proposed.

(f) Is authorization by voters required? If election has already been held, submit certified copies of all proceedings taken in respect of such election, including authorization for election, notice, ballot, and certificate of result.

(g) In authorizing construction of proposed project and issuance of proposed obligations, does applicant act by ordinance or resolution?

(h) Is it necessary for such ordinance or resolution to be published before taking effect?

(i) Is it necessary for applicant to obtain any franchise or other permit from any public agency or other authority to construct and/or operate the proposed project? If so, state character of franchise or permit, from whom necessary, what steps, if any, have been taken to obtain it, or if not yet obtained, what steps must be taken and within what approximate time may such franchise or permit be obtained.

(j) May applicant sell proposed obligations at private sale, or is it necessary to sell such obligations at an advertised public sale? If public sale is required, state manner and time required for giving notice.

(k) State maximum rate of interest which proposed obligations may bear.

(l) State minimum price for which proposed obligations may be sold.

(m) Is it necessary for applicant first to offer proposed obligations to any other authority before such obligations may be sold to the public? If so, state to whom, giving reference to appropriate section of statute.

(n) Give reference to provisions of statute regulating maturities of obligations, stating time within which first installment of obligations must become due and maximum period within which last installment must become due.

(o) If statute regulates amount of obligations falling due in any given year, state restrictions, giving reference to appropriate section of the statute.

(p) Are proposed obligations negotiable instruments under the negotiable instruments law?

(q) Are the proposed obligations legal investments for administrators, trustees, and other fiduciaries, and for savings banks and insurance companies organized under the laws of the State in which the applicant is situated?

(r) May proposed obligations be made registerable as to principal only or as to both principal and interest?

(s) Do proposed obligations and the proceedings for their issuance have to be approved by the attorney general of the State or any other authority? If so, by whom?

(t) Are the proposed obligations required to be registered in the office of any State, county, municipal or other authority prior to their issuance?

(u) Will proposed obligations be exempt from all taxation (except estate, inheritance, and gift taxes) under the laws now in force of the State in which the applicant is situated?

(v) Will interest to be paid on proposed obligations be exempt from all taxation (except estate, inheritance, and gift taxes) now or hereafter imposed by the United States under the Constitution now in force?

(w) Is any litigation now in process, pending or threatened (either in State, local, or Federal courts), in anywise affecting the corporate existence of the applicant, the authorization, execution, sale and delivery of the proposed obligations, the construction of the proposed project, the levy and collection of taxes to pay the principal of and interest on the proposed obligations, the boundaries of the applicant, or the incumbency of any of its officers?

(x) Will it be necessary to obtain property by condemnation proceedings, or otherwise? Give reference to law authorizing proposed proceedings and state approximately how long it will take to acquire such property.

(y) Give name and address of bond counsel of recognized national standing who will furnish opinion on proposed obligations.

(z) If the proposed securities are what are generally known as revenue bonds, and are payable only from the revenues to be derived from the operation of the project, the holder thereof having no recourse for payment to the general taxing power of the municipality, then—

(1) Does the issuance of such securities constitute the incurring of a debt by the municipality within any State constitutional or statutory provision or limitation? Has the court of last resort in the State in which the applicant is situated passed upon this question? If so, cite decisions.

(2) Has the court of last resort in the State in which the applicant is situated held that revenue bonds or other like obligations issued for the construction of extensions or improvements to an existing municipally-owned revenue producing utility, when made payable from and secured by a pledge of the revenues of the existing utility as extended or improved, constitute debt? If so, cite decisions.

(3) Are proposed obligations payable from the gross or net revenues of the existing utility as extended or improved, or are such obligations payable from the gross or net revenues definitely allocable to or shown to be derivable from the proposed extensions or improvements to the existing utility?

(4) In the event of failure on the part of the municipality to perform the covenants and agreements to be set forth in the ordinance or resolution authorizing the issuance of the proposed obligations, what will be the remedies of the holder of such obligations?

(5) If the proposed obligations are additionally secured by a mortgage or trust indenture, has the court of last resort of the State in which the applicant is situated passed upon the power of the applicant to make such a mortgage or trust indenture? If so, cite decisions.

(aa) If the proposed securities are what are generally known as "special assessment bonds" or other like obligations, then state:

(1) How are such special assessments fixed and when do they become final?

(2) In the event of default in the payment of the principal of or interest upon such obligations, what are the remedies of holder?

(3) What character of lien have such special assessments on the properties benefited and is this lien superior or junior to the lien of other taxes?

(4) Is there any additional security for the payment of such obligations other than the special assessments?

(5) Does the issuance of such obligations constitute the incurring of a debt?

(bb) If the applicant has no power to issue general obligations or special revenue bonds, give reference to provisions of general, special, or local laws empowering applicant to convey site upon which project is to be constructed to the United States and enter into a lease with the United States relative to the project and obligate itself to pay rental therefor.

Contractor Finances Work In St. John Harbor

A departure from ordinary practice is revealed in connection with a contract recently awarded for the reconstruction of part of the Saint John, N. B., harbor. The general contractors, the Saint John Drydock and Shipbuilding Company, has undertaken to carry the cost of the work for three years, or in other words, to advance the necessary capital to the Dominion Government, with provision for repayment after three years.

Buffalo Sewage Plans To Be Revised

George B. Gascoigne, consulting engineer, Cleveland, Ohio, was authorized by the special public works committee of the common council, Buffalo, N. Y., on July 24, to revise the so-called Gascoigne-Tolles plan for new sewers and sewerage disposal plants as a possible federal emergency work relief measure. Mr. Gascoigne said that when he made his estimate of $23,000,000 in 1929 for the project, that the estimate was based on the higher labor and material costs then prevailing and that now the project probably would cost considerably less.

Contracts and Capital

CONSTRUCTION
(Thousands of Dollars)

	Weekly August, 1932	Average Four Weeks	Week Aug. 10, 1933
Federal Government	$4,177	$322	$296
State and municipal	12,440	5,784	4,554
Total public	$16,617	$6,106	$4,850
Total private	3,792	7,089	8,759
Week's total	$20,409	$13,195	$13,609
Cumulative to date:			
1932		$759,390	
1933		521,796	

NEW PRODUCTIVE CAPITAL
(Thousands of Dollars)

	Weekly Aug., 1932	Average Four Weeks	Week Aug. 5, 1933
New capital issues	$5,400	$77,190	$128,077
Cumulative to date:			
1932		$501,400	
1933		480,193	

ENR Cost and Volume Index

E.N.-R. Cost		E.N.-R. Volume	
August, 1933	167.00	July, 1933	65
July, 1933	165.50	June, 1933	109
August, 1932	156.80	July, 1932	146
1932 (Average)	156.97	1932 (Average)	127
1931 (Average)	181.35	1931 (Average)	220
1930 (Average)	202.35	1930 (Average)	260
1913 Average	100		

Obituary

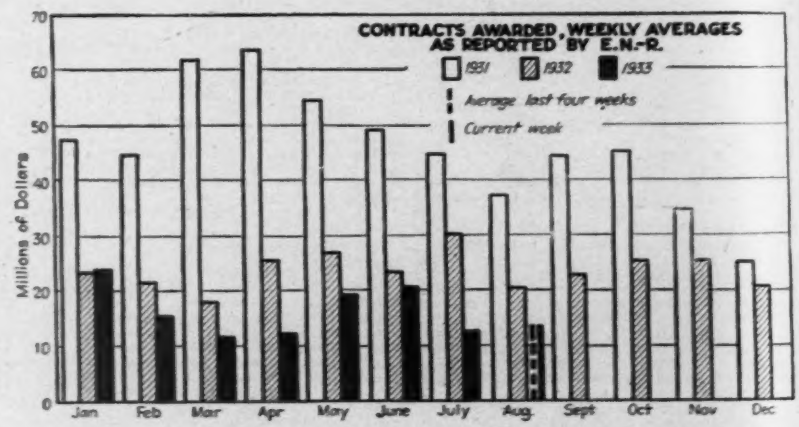
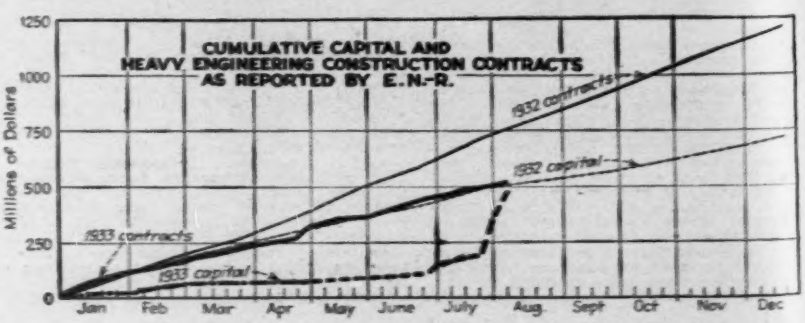
OLIVER C. WILKES, widely known architect and builder, Buffalo, N. Y., died in that city July 23, at the age of 84 years after a long illness. He was the designer and builder of a number of large churches in Buffalo. He is survived by one daughter.

EARLE P. LUCE, formerly county engineer of Duval County, Fla., died on July 5 at the age of 40 following an emergency operation. He had been ill for several months. Mr. Luce was graduated from the Michigan State College of Mines. He went to Jacksonville in 1918 following service in the army during the war.

FRANK D. HUDGINS, aged 55, senior highway engineer in charge of the Little Rock, Arkansas, office of the Federal Bureau of Public Roads, died at his home in that city, July 25, after an illness which began last February. Mr. Hudgins came to Arkansas from South Dakota, where he was at one time state highway engineer. He also had been connected with many other engineering projects.

HARRY J. FINEBAUM, consulting engineer of New York City, was killed in an automobile accident on July 4 in Florida. Mr. Finebaum was 40 years old. He was born in London, came to this country in his youth and was graduated from Cooper Union and Brooklyn Polytechnic Institute. At the time of his death he was under engagement as supervising and consulting engineer in connection with contract work in government flood control projects.

Engineering Contracts and Capital

HEAVY CONSTRUCTION has not as yet felt any effects of the national public works program according to contract award figures. Contracts reported last week showed a slight gain over those of the past two weeks, yet remain close to the average of the past month. The total of contracts reported is $13,609,000, as compared with $12,189,000 for the previous week. The average of the past four weeks is $13,195,000. A lag in public works, coupled with a $3,500,000 contract for a new railroad terminal at Houston and a million-dollar brewery in New York City, raised private contracts to nearly double that of public works for the past week, the totals being $8,759,000 and $4,850,000, respectively. Highway contracts held to the low average of the past few weeks, amounting to $2,487,000. Federal government work also remained low, with only $296,000 reported in contracts let. Meanwhile the list of proposed projects, both large and small, continues to grow. While many of these prospects are only in the planning stage, bids are being asked for on others for September and October openings.

Considering the definite allotments to the various states for highway construction, new capital available for construction has more than doubled in the past two weeks. Up to two weeks ago cumulative new productive capital for the year was only $183,000,000, about a third of that of last year. Now the cumulative total stands at $480,000,000. An encouraging sign in the municipal financing situation is the appearance of bond sale schedules for municipal construction projects. For over a year municipal bond sales have been almost exclusively for relief work or for general financing.